WATER MANAGEMENT PLANNING FOR

THE WESTERN C-51 BASIN.

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Water Management Planning for the Western C-51 Basin, Palm Beach County, Florida

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PART I

PROJECT BACKGROUND AND DESCRIPTION OF THE PROBLEM

A. HISTORY OF THE PROJECT

1. **Study Authority.** The construction of works for the West Palm Beach Canal (C-51) basin by the United States Army Corps of Engineers (USCOE) was authorized by congressional resolutions of 1948, 1962, and 1968. The purpose of these plans was to improve management of the C-51 and other Florida east coast canal basins to provide flood protection, water supply, and environmental benefits. The present investigation is being conducted by the South Florida Water Management District (SFWMD), in coordination with the USCOE, as an extension of the originally authorized work. The investigation includes consideration of structural and other appropriate measures to improve the future flood control capabilities within Palm Beach County and to protect the environmental resources of the Water Conservation Areas (WCA's).

2. **Public Concerns.** Land owners in the western portion of the C-51 basin have expressed continuing concern regarding the adequacy of flood protection in this basin. Recent occurrences of heavy rainfall have resulted in severe localized flooding which has posed a threat to property. Citizens in the eastern portion of the C-51 basin have also expressed similar concern, because portions of this basin have been subjected to flooding during the past two years. In some cases, these problems will be alleviated by the completion of the new coastal water control structure (S-155) which is currently under construction. However, during a severe basin-wide storm, the need to discharge flood waters from the western basin may exceed the design capacity of S-155 and result in higher stages and extended duration of flooding in the eastern basin.

A number of environmental organizations, the United States Fish and Wildlife Service, the Florida Department of Environmental Regulation, and the Florida Game and Freshwater Fish Commission, have expressed concern regarding the impact of pumping excess amounts of flood waters into the Water Conservation Area system. Two primary concerns have been expressed as follows: a) the WCA's have remained at high water level stage conditions for extended periods of time during recent years. These high stage conditions, especially in WCA-2A, have resulted in adverse changes in vegetation and wildlife habitats, causing the drowning of tree islands and the accumulation of flocculent layers of organic material on the bottom of the marshes, and b) the quality of water that will be discharged into the WCA's during storm events is of concern due to the presence of nutrients and pollutants from urban and agricultural runoff. Continual discharges of this storm water may have adverse effects on fish, wildlife, and plant communities within the Water Conservation Areas.

3. Planning Objectives. The principal objective of this study is to examine alternatives for improving water management conditions in western Palm Beach County to protect existing development from flood damages. A second objective is to determine the impacts of improved water management in the western basin on flood conditions in the eastern basin. The third objective is to determine whether backpumping of the C-51 basin will have any significant adverse impacts on environmental resources within the region and to identify plans which will minimize impacts, if any.

B. STUDY AREA

1. **Location, Boundaries, and Description.** The West Palm Beach Canal extends from Lake Okeechobee south and east to the coastline near West Palm Beach. The study area is the drainage basin of C-51, which is the designation given by the USCOE to the easternmost segment of the West Palm Beach Canal. C-51 extends from the northern end of WCA-1 to Lake Worth. The C-51 basin occupies approximately 174 square miles in central and northern Palm Beach County (Figure 1). The western portion of the C-51 basin covers about 100 square miles and generally includes the area north to the M Canal, south to the ACME Drainage District, west to WCA-1 and the L-8 Canal, and east to State Road 7 (SR7) and the eastern boundary of Wellington. The eastern basin lies generally north of Lake Worth Road, south of Okeechobeee Blvd and east of SR7 (Figure 2).

2. **Land Use and Economy.** The general pattern of land use in the C-51 basin was surveyed by the SFWMD using a 1979-1980 data base. It was suspected, however, that substantial changes had occurred in the basin during the last three years; therefore, these data were updated by examination of records of Palm Beach County, Royal Palm Beach and Wellington developments, U.S. Census data for 1980, and the SFWMD permit files.

The C-51 basin is an area of mixed land uses. The western portion of the basin contains approximately 71,000 acres, of which 13,300 acres are under cultivation for pasture, citrus, or sugarcane. Portions of the basin are currently in truck crops; however, this acreage varies from year to year and much of it will probably be converted to residential use within the foreseeable future. Pasture accounts for a large proportion of cultivated land in the basin (approximately 4,200 acres). Other agricultural land uses include approximately 6,300 acres of citrus and 2,800 acres of sugarcane. In addition, the basin contains two urbanized areas--Wellington and Royal Palm Beach--and many suburban areas. The eastern portion of the basin has been urbanized for many years. The western part of the basin has become increasingly urbanized as the existing residential developments expand into areas that were previously either agricultural or undeveloped. The count of residential units in the eastern and western basins is shown in Table 1.

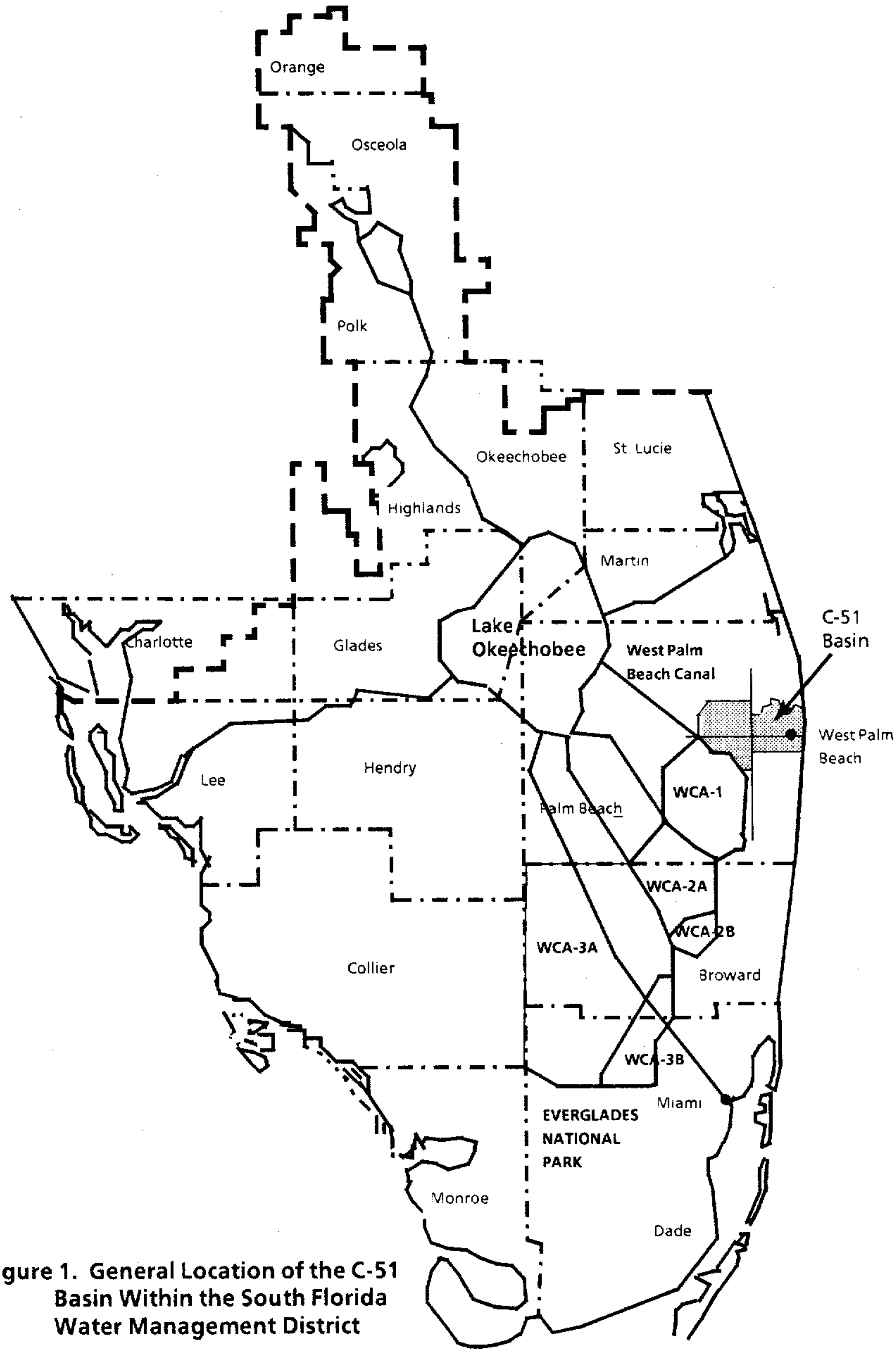

Figure 1. General Location of the C-51 Basin Within the South Florida Water Management District

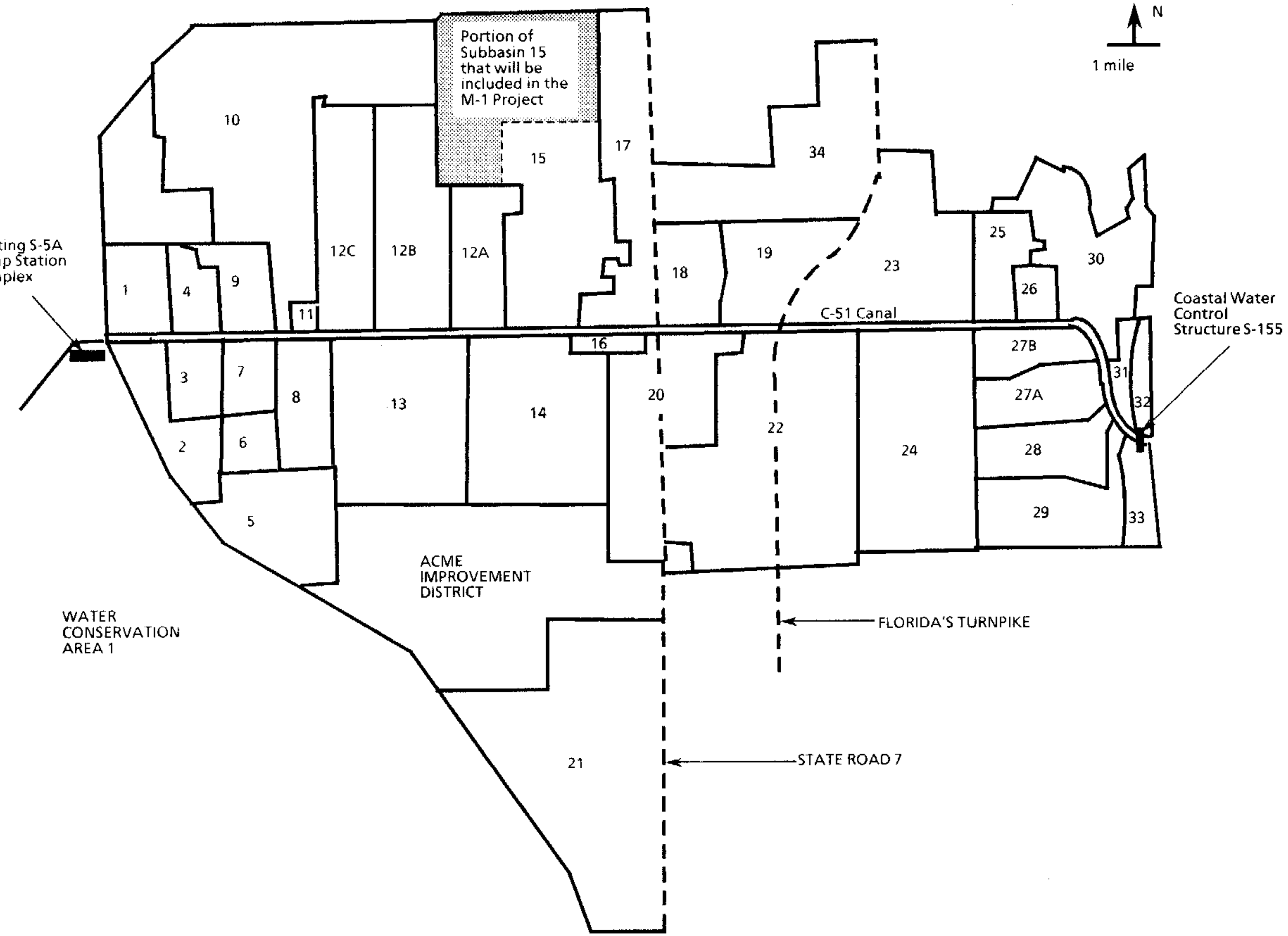

Figure 2. Major Features of the C-51 Basin in Palm Beach County, Florida

Table 1. Housing Units in the C-51 Basin (1980)

	Total Units*	Owner Occupied	Rental Units
Eastern Basin	52,407	31,764	15,025
Western Basin	14,071	8,115	2,290
TOTAL IN BASIN	66,478	39,879	17,315
TOTAL IN COUNTY	295,664	171,736	62,603

*Includes seasonal units so that total units exceed owner-occupied and rental units

Table 1 shows that the C-51 basin included nearly 25% of the dwelling units in Palm Beach County in 1980. The agricultural lands in the western C-51 basin contain high-value crops. Thus, the C-51 basin accounts for a significant proportion of the economic activity in Palm Beach County.

3. Water Management.

a. C-51 Basin. Water management in the C-51 basin is largely controlled by the SFWMD through operation of major pumping facilities and structures, and conveyance through the major SFWMD canals. A number of local drainage districts, private landowners, and municipalities operate smaller pumps and water control structures of the secondary drainage system under permits from the SFWMD.

The facilities that are maintained and operated by the SFWMD include a complex of control structures and a basin discharge structure (S-155) that is scheduled to replace the Palm Beach Locks by October 1984. The canal (C-51) is approximately 24 miles long and extends from the L-8 divide structure (S-5AE) to a point downstream of S-155. The drainage area of this portion of the canal has been divided into two basins, with the dividing point at State Road 7 (See Figure 2). Water stages in C-51 generally range from 8 ft NGVD at the coastal structure at Lake Worth to 13.5 ft NGVD at S-5AE. Flood Stages in the canal, under existing conditions, could range from 12 ft NGVD at S-155 to 18.3 ft NGVD at SR7 during a 1-in-100 year storm event. Peak stages would drop to 17.8 ft NGVD at S-5AE, due to overbank flow into low-lying basins south of the canal.

The water control structure at the eastern end of C-51 is the Palm Beach Lock and Spillway, which regulates flow out of the canal and into the

Intracoastal Waterway at Lake Worth. This structure consists of a gated spillway and an eight-barrelled box culvert that is controlled manually by stop logs. A new structure (S-155, as indicated above) that can be operated automatically is under construction.

b. WCA-1. Water Conservation Area 1 occupies an area of 221 sq mi. Diking of the area was completed in 1956. This area is used as a water storage facility and as a means of perpetuating a portion of the original Everglades. Ground elevations range from 17 ft NGVD at the northern end to 12 ft NGVD at the southern end. Water stages in WCA-1 are regulated between 14 ft and 17 ft annually. Vegetation in the area consists of extensive sawgrass marshes, wet prairies, sloughs, and numerous tree islands. The area is leased to the United States Fish and Wildlife Service (USFWS) and is designated as the Loxahatchee Wildlife Refuge. Water enters WCA-1 from rainfall and from S-5A and S-6 pump stations which discharge runoff water from the Everglades Agricultural Area. Three private pump stations, which operate under permits from the SFWMD, also pump water into WCA-1. Water is generally released out of WCA-1 into WCA-2 in order to maintain appropriate stages. WCA-1 also supplies water to the east coast via S-39 to the Hillsboro Canal for irrigation and maintenance of appropriate groundwater stages. Two local drainage districts withdraw water directly from this area, primarily for irrigation.

4. **Hydrology.** The C-51 basin has been divided into a number of hydrologic sub-basins as shown in Figure 2. Major inflows from these sub-basins to the canals are generally regulated through the operation of pumps or water control structures. A detailed hydrologic analysis of these sub-basins, under various conditions of rainfall, land use, and water management scenarios, is presented in Appendix A. Under current conditions, substantial variation exists in the ability of existing facilities to remove flood waters. Although the permit requirements of the SFWMD restrict how much water can be removed from certain areas during flood conditions, substantially more water can be removed from some of these areas during non-flood conditions. Analyses of existing conditions in the basin were based primarily on how the system that is now in place would realistically be operated during a flood. The proposed changes that will divert water from the M-1 sub-basin away from C-51 and into the L-8 Canal were considered as a separate case in these analyses. Analyses of future conditions, with backpumping facilities in

place, were conducted both with and without the assumption that the operation of the water control structure at the outlet of Royal Palm Beach would be restricted to the limits imposed by SFWMD permit. These analyses were conducted to determine whether flood damages in Royal Palm Beach could be significantly reduced if the structure were allowed to operate at its full capacity during storm events.

5. Water Quality. An analysis of water quality conditions in the C-51 basin, and the impacts that runoff from this basin may have on receiving waters of WCA-1, is presented in Appendix B. Water quality data for the C-51 basin were obtained from studies conducted by the SFWMD and private consultants. Water quality varies greatly depending on land use patterns in adjacent uplands and the distribution of rainfall and runoff. Agricultural practices are a major potential source of nutrients in runoff water from this basin. In addition, the basin contains a number of sites that are used for disposal of sewage sludge. The use of pesticides by agricultural interests in the basin poses another potential water quality problem. Runoff water quality may improve somewhat as the basin becomes increasingly developed, due to drainage systems that are designed to mitigate pollution from runoff. A more complete summary of the water quality studies is presented in the portion of Part III that considers water quality impacts of backpumping.

6. Natural Features. The analysis of 1979-1980 land use patterns indicated that large tracts of land within the C-51 basin were undeveloped. Conditions have changed since the time of this survey and much additional land has been placed under development and is no longer vacant. The existing patterns of land cover in the C-51 basin have been heavily impacted by general lowering of the water table during recent years as well as by the impacts of fire and invasion of exotic plant species. Historically, the area was covered by extensive wetlands, including cypress forests, wet prairies, and sawgrass marshes. Upland areas contained pine-palmetto flatwoods and hardwood hammocks. Some remnants of these natural areas remain in the C-51 basin, but the majority of the basin has been drained and managed for agricultural use or is in the process of being subdivided and sold for residential or agricultural development.

C. POTENTIAL IMPACTS OF FLOOD EVENTS UNDER EXISTING CONDITIONS

1. Basis of Hydrologic Analyses. Potential impacts of various flood events were determined by the application of storm routing models of the C-51 basin. Hydrological models, based on standard engineering procedures for flood and drainage analyses, were modified for use in this study and were applied based on specific data for the C-51 basin. A detailed description of the modeling procedures, methods, and assumptions used in this study is presented in Appendix A. A summary of the results of this modeling analysis for existing conditions in the basin is presented in this section. The results of the hydrologic analyses were used as the basis of damage assessment analyses, analyses of water quality impacts, and evaluation of environmental impacts.

Rapid urbanization of the C-51 basin during the past several years has created increasing concern that flooding may occur in the western portion, especially in the Village of Royal Palm Beach. Additional damages may also occur in the eastern basin, in areas such as Lake Clarke Shores during severe storms, due to the additional runoff that is generated in the western basin . Backpumping of excess runoff from the western C-51 basin to WCA-1 would provide some protection to existing properties from flood damage. Several case studies, both with and without backpumping, were evaluated for this report.

a. Land Use Data. In order to define the magnitude of the existing problem and to determine whether a project is necessary, existing land use was documented within the basin. These data were used to provide an estimate of potential damages and as a data base for the modeling studies. "Existing land use" was defined from 1979-1980 aerial photographs and was supplemented with data from Palm Beach County building permit records. "Committed land use" refers to areas where no existing land use data were available, but where a development permit had been issued to a developer from a government entity such as theSFWMD. The permit information, as of February 1983, was used in this study. The "future land use" conditions used in the backpumping plans were based on projections from local government comprehensive plans as supplemented by data from the SFWMD permit files

b. Rainfall and Runoff Conditions. Rainfall and other hydrologic parameters used as the basis for these studies are described in detail in

Appendix A of this report. The amounts of rainfall for the 1-in-10, 1-in-25, and 1-in-100 year storm events were compiled from a number of sources, but as much as possible, local rainfall patterns were used. The peak rainfalls used for analysis of flooding in this study were:

Storm Return Frequency (Years)	Peak 1-Day Rainfall (Inches)	Peak 5-Day Rainfall (Inches)
10	8.5	12.70
25	10.5	15.47
100	13.5	19.59

In each, the peak 5-day rainfall was used as the first 5 days in September, with the peak 1-day rainfall occurring on the fifth day. With the wet antecedent soil conditions, created during the first four days of the design storm, essentially all of the rainfall on the fifth day would become direct runoff and several inches of water would pond on top of the ground throughout the basin during the heaviest storm period.

c. <u>The M-1 and M-2 Projects.</u> Several rural residential projects are in various stages of development in the northern area of the western portion of the C-51 basin. Two such projects, M-1 and M-2 , are being constructed by the Indian Trail Water Control District. Both of these projects consist of 1.25 acre homesites. Seven square miles (called the Royal Palm acreage or M-1 acreage) of the 28 sq mi area in the M-1 project are currently draining into C-51 via the main canal of Royal Palm Beach (See Figure 2). This seven square miles will ultimately join the remainder of the M-1 area and drain north and west into the L- 8 canal upon completion of the M-1 project. The bonds for this project have been sold and construction of the drainage facilities is expected to be completed within two years. For the purpose of analysis of existing conditions, it was assumed that this project has not been completed. Additional scenarios were run to determine the impacts of completion of the M-1 Project.

2. Hydrologic Analysis of Existing and Committed Land Use Conditions with Royal Palm Acreage Area Included in The Royal Palm Beach Sub-basin.

a. Impacts in the Western Basin. The 1-in-10, 1-in-25, and 1-in-100 year storm events were evaluated in the C-51 basin. Sub-basins 2, 3, 5, and 6, which are agricultural areas in the westernmost portion of the basin (Figure 2), would become water storage areas and receive overflow from C-51 during all three design storm events. Sub-basins 7 and 8 and the ACME Drainage District would receive floodwaters from C-51 during the 1-in-100 year storm. Sub-basins 1, 4, and 9 would receive runoff from areas to the north, such as the Callery Judge Citrus Groves and the Deer Run and Dellwood developments, etc. The water stage in C-51 would peak at a location near SR7.

Results of flood duration analyses in the western C-51 basin for the design storms indicate that peak flood stages in Royal Palm Beach (sub-basin 15) would be as follows:

Storm Return Frequency (Years)	Peak Flood Stage (Ft.NGVD)
10	19.35
25	19.70
100	20.10

Figure 3 shows peak flood stages that are expected to occur in the western sub-basins of the C-51 watershed during the 1-in-10 and 1-in-100 year storm events.

b. Impacts in the Eastern Basin. Expected peak water levels in the eastern reaches of the C-51 canal at selected locations are:

Storm Return Frequency (Years)	Peak Stage @SR7 (Ft., NGVD)	Peak Stage @Summit Blvd. (Ft.,NGVD)	Peak Stage @Forest Hill Blvd (Ft., NGVD)
10	17.96	12.25	11.75
25	18.10	12.70	12.20
100	18.26	13.30	12.80

Since the general ground elevation near SR7 ranges from 16 to 18 Ft. NGVD, the ground and secondary roads in this area would experience

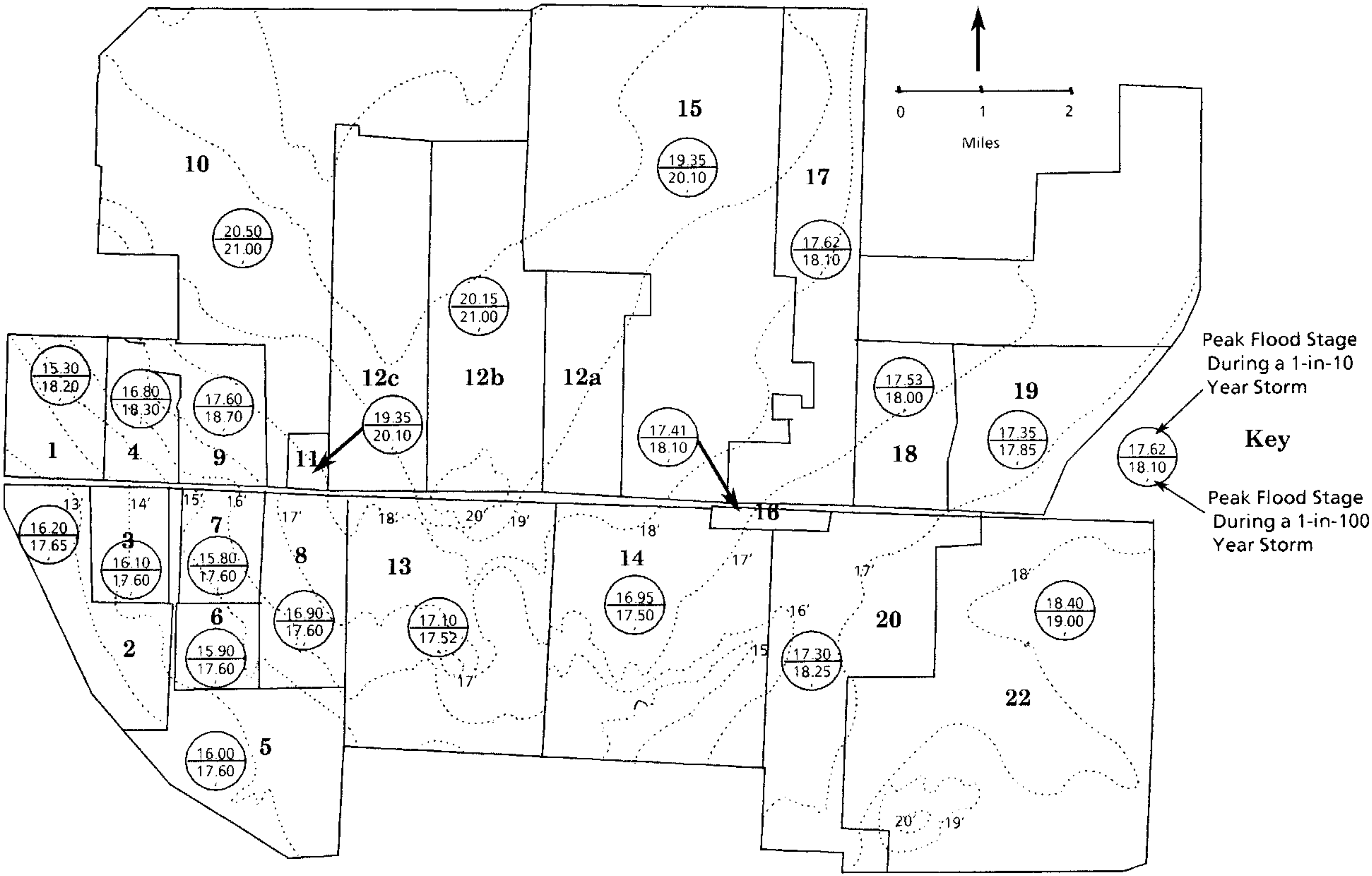

Figure 3. Peak Flood Stages for 1-in-10 and 1-in-100 Year Storm Events (ft NGVD) in the Western Subbasins of the C-51 Basin--Existing Conditions.

significant flooding during all of the design storms. Houses with floor elevations below 13.50 Ft. NGVD in the Summit Blvd. area would probably be flooded during the 1-in-100 year storm.

3. Damage Assessment Analyses of Existing Conditions. A flood damage assessment study was conducted to identify significant potential flood damages in the C-51 basin. The study focused on those sub-basins and damage categories in which potential damages could be either confirmed or refuted in the most cost-effective manner. No attempt was made to measure damages for sub-basins and categories which were thought to be small or difficult to determine; therefore, all damage estimates must be viewed as partial, and hence underestimate the true damage potential.

Damages were estimated for existing agricultural and urban development. The types of flood damages that were expected in the C-51 basin include damages to building structures, building contents, urban landscaping, streets, and crops. Table 2 indicates how each of these types of damages was considered in this study.

The losses that could be expected as the result of floods were determined for the 1-in-5, 1-in-10, and 1-in-100 year storm events. Estimates were summarized separately for the eastern and western basins and for urban and agricultural losses.

Results of this initial damage assessment analysis for the existing conditions in the basin are presented in Table 3. This preliminary analysis indicated that a substantial flood damage threat presently exists in the C-51 basin. Damages in the 1-in-100-year storm could exceed $26 million and damages in the 1-in-10 year storm could exceed $13 million. Significant factors in deciding whether to construct a project, in additition to the magnitude of the existing problem, are the benefits that would be derived. In this case the benefits would accrue as a reduction of expected damages due to changes in the hydrology of the basin.

TABLE 2. DAMAGE CATEGORIES AND DAMAGE MEASUREMENTS ANALYZED IN THE C-5l BASIN

Damage Category	Damage Coverage
Building Structures	Estimates were completed for residential single family and multi-family units but not commercial and industrial units.
Content Damage	Estimates were completed for residential single family and multi-family units but not commercial and industrial units.
Lawns, Trees, Shrubs, Sidewalks, and Driveways	These were not estimated because they largely depend on durations of flooding, and the flood periods in almost all situations would not be sufficient to create significant damages.
Streets	These were not estimated because, while they depend on durations, the durations in almost all cases were not long enough to produce significant damages.
Citrus	Damages were estimated.
Improved Pasture	Damages were estimated.
Sugar Cane	Damages were estimated.
Truck Crops	Damages were not estimated because it was felt that most severe storms would occur during the wet season when few truck crops will be planted. In addition, the acreage and locations of truck crops in this basin varies significantly from year to year.
Nurseries	Damages were not estimated because relatively few acres were identified and because of further difficulties in determining the particular types of nurseries and their susceptibility to damages from inundations.

TABLE 3. PRELIMINARY DAMAGE ASSESSMENT-EXISTING CONDITIONS

Estimated Damages ($1000's) for Storm Events

	1-in-5	1-in10	1-in-100
Western Basin			
Agricultural	1905	10,949	17,224
Urban	0	2,433	8,813
Eastern Basin			
Urban	0	13	111
TOTAL	1905	13,395	26,148

PART II

ALTERNATIVES

A. GENERAL

This section identifies and describes the range of alternative plans, including structural, non-structural, and operational methods, that were considered to address the problems of the C-51 basin . The alternative plans were developed to meet the immediately identifed objective of flood protection, while continuing to address environmental and other regional water management objectives. The merits and liabilities of each of these approaches were evaluated to determine the best overall strategy that would give balanced consideration to all objectives. Based on this initial screening, six scenarios were considered for detailed analysis.

B. PLANS ASSIGNED LOW PRIORITY AND NOT STUDIED IN DETAIL

A number of planning options that were initially considered for use in the C-51 basin were assigned low priorities because preliminary analyses indicated that they had excessive costs or adverse environmental and social impacts. The basic features of these plans are described here because they may be reconsidered in the future.

1. Structural Options

a. Increased Storage in the Corbett Area. The J.W. Corbett Wildlife Management Area is a large tract of land in northwestern Palm Beach County that is managed by the Florida Game and Freshwater Fish Commission as a wildlife preserve. The area is entirely undeveloped and is maintained as a mixture of native wetland and terrestrial habitats for deer, feral hogs, game birds, and fish. The current management scheme calls for maintenance of a maximum water elevation of approximately 16 ft NGVD during the wet season and maintenance of 14 ft NGVD during the dry season. If higher transient stages could be held in this area, substantial additional storage capacity for floodwaters from the C-51 basin would be available. Use of this area would require improvement of adjacent conveyance canals, a system of retention levees, and pumping capacity to lift water into the area, as well as raising the elevation of at least one major road, SR710. This plan was initially rejected for three reasons: 1) it was felt that soils of the area were sufficiently porous and water would "leak " into adjacent basins; 2) even If the water could be retained in the Corbett Area, this additional water would have adverse impacts on the existing wildlife; and 3) the costs of constructing levees around an area of this size was felt to be prohibitive.

b. New Water Conservation Areas. Undeveloped lands in and near the C-51 basin were examined to determine whether any large tracts of land could be used to create a new Water Conservation Area. However, due to the value of land in these areas, it was felt that no suitable undeveloped wetlands remained in large tracts outside of the Corbett Area. Some portions of the western C-51 basin that had been wetlands, in areas such as Fox Trail and portions of Wellington, have been severely impacted by drainage in recent years and are currently under intense pressure for agricultural and estate-density land development. The costs of land purchase added to the costs of construction of pumps and levees were considered to be prohibitive.

c. Flowage Easements. The possibility of purchasing flowage (flooding) easements on agricultural lands was considered. Approximately 8,500 acres of agricultural land are involved in sub-basins 1, 2, 3, 4, 5, 6, 7, and 8. SFWMD experience has shown that, as a minimum, a flowage easement will cost 90% of the land purchase cost. Therefore, it was estimated that flowage easements would cost approximately $38.2 million for affected agricultural lands. Including the affected residential houses would add $13.8 million to the cost of this alternative.

d. Increased Discharge to Eastern Basins. In the 1970's the new structure at S-155 was designed to meet anticipated levels of discharge from the eastern basin, with the assumption that the western basin would eventually be backpumped. However, the western basin is still discharging to the east under existing conditions, and the projected levels of discharge upon which the design of S-155 was based, have been reached. The costs of further improvements to this structure, and associated canal right-of-way purchases through urbanized areas, are estimated to be in excess of $35 million, so it would be most practical to find some alternative means to provide for flood protection in the western basin.

e. Connection to the C-18 Basin. The original plans for backpumping the C-51 basin were developed in 1968 and 1973 and considered the potential for connecting the L-8 and northern C-51 drainage basins to the Loxahatcheee Slough and C-18 system. The possibility of constructing such connections in the L-8 basin will be considered in a subsequent study to be undertaken by the SFWMD. The principal benefit of such a connection would be water supply, with only minimal flood protection.

2. **Nonstructural and Minor Structural Options.** Several methods that involve small-scale structural changes within the system or are nonstructural in nature were considered for application within the C-51 basin. These methods may be applied to decrease the duration and extent of flooding and the amount of damage that may be sustained during a flood event. Many of these options would be best implemented at the local or county levels or through the SFWMD's regulatory programs. The methods considered are briefly summarized as follows:

a. Modification of Water Management Practices. The potential for flood damage within the basin may be substantially reduced by developing additional on-site retention capabilities for residential and agricultural areas, and undeveloped tracts within the basin. Existing lakes and canal systems could be enlarged to provide additional storage. Conditions of surface water management permits in this basin could be modified to require additional onsite retention of runoff and reduce the magnitude and duration of peak discharges to C-51. Undeveloped or agricultural lands within the basin could be used intermittently as water detention or retention areas. Increased water storage capacity may occur in the form of decreased rates or amounts of water removal provided by the system, or by pumping water onto the property. Methods such as the purchase of easements or the transfer of development rights may be applied to prevent or restrict the development of flood prone or retention areas (see item 1.c., above).

b. Purchase of Flood-Prone Areas. Those portions of the C-51 basin most prone to flooding are identified in this study. The Water Management District or another state or Federal agency could purchase these lands or appropriate easements. Flood prone lands within the basin have been identified on the basis of topographic and hydrologic data. Major portions of the basin have been subdivided and sold in small, 5 to 10 acre parcels. The value of this land has increased dramatically in recent years due to rapid development, speculative buying, and the construction of large, estate-type housing in the basin.

c. Restrictions on New Construction. Analysis of existing conditions in the C-51 basin indicates that additional flood protection is needed to protect buildings and land uses that are already in place. New building standards should be developed and implemented so that this problem does not become worse. The better estimates of the flooding threat in the C-51 basin, which are provided by this report, should be incorporated into new building standards so that future structures and developments are constructed well above flood hazard elevations.

d. Flood-Proofing of Existing Development. Protective structural changes, such as the installation of small pumps and/or construction of low-level barriers to restrict water flow, could be made by individual property owners to prevent or reduce damages to homesites or developments. Cost estimates for construction of a low-level, three-foot high barrier around an individual structure have indicated that such measures could cost 0.5% of the value of the structure per year, or about $500 per year for a $100,000 home. Floodproofing may be particularly appropriate in areas that would still experience significant flooding even if a structural flood control plan were implemented in the western C-51 basin.

C. PLANS STUDIED IN DETAIL

A number of scenarios for the C-51 basin were analyzed, based on several structural features. The impacts of these scenarios on environmental conditions in the WCA's were also evaluated. Operational impacts of the scenarios and possible operational changes that could be made to minimize impacts were also considered.

1. Major Structural Design Features. Four basic structural elements were considered as integral to the plans to modify the C-51 basin.

a. New Pump Station (S-319) at the West End of C-51. A 5000 cfs pump station (S-319) was the central element of the original 1972 USCOE proposal to backpump the western C-51 basin. In 1982, the SFWMD examined the possibility of placing a series of smaller pumps, with a total capacity of 550 CFS, at this location to provide additional water supply to the WCA's. A pump station at this location could be operated to provide additional flood protection to the western C-51 basin during wet periods and could be operated during dry periods to enhance the regional water supply. A pump of 3400 cfs capacity would be necessary to provide minimum flood protection requirements of the western basin.

b. Water Control Structure at State Road 7 (SR7). The backpumping plans require that the existing water management system in the C-51 basin be modified to divide the basin at SR7 between the eastern and western basins. A new water control structure, S-155A, would be constructed at this location. This structure would allow for flow of water from the western basin to the east under less than design storm conditions. During a design storm, this structure could be closed so the runoff that would be generated in the western basin could be backpumped into WCA-1. A range of sizes for this structure, from 100 cfs to 1000 cfs discharge capacity, was evaluated for the backpumping plan. In general, the smaller structures were less expensive to build and operate, but they would require more frequent backpumping into WCA-1, which was a special item of concern in this study.

c. Canal Improvements in the Eastern and Western Basins. The original USCOE studies of the C-51 basin identified the need to improve the conveyance capacity of C-51 in the reach east of SR7. These improvements are necessary to allow the new structure at the Intracoastal Waterway, S-155, to function at its design capacity and provide adequate flood protection to the eastern basin. All plans for backpumping are based on the assumption that these improvements to the canal are completed as planned. In addition, the SFWMD conducted a survey of the western reach of C-51 in the spring of 1983 to determine the ability of this portion of the canal to convey flood waters from the western basin back to WCA-1. This survey indicated that some improvements to the western end of the canal were necessary in order to obtain the full 3400 cfs conveyance capacity.

d. Diversion of Surface Water Runoff from the C-51 Basin to the L-8 Basin. This will be accomplished by construction and operation of a local drainage project that is underway by the Indian Trail Water Control District. Runoff from approximately 7 sq mi of sub-basin 15, called the M-1 acreage, will be diverted to the L-8 basin by this project. This runoff will be contained in a detention area and later discharged to the L-8 Canal The diversion will provide improved flood protection to the northern areas of Royal Palm Beach and will decrease the extent and duration of flooding throughout the remainder of Royal Palm Beach.

In addition to the four elements described above, another structural alternative was analyzed as a method for reducing flood damages within the Royal Palm Beach area:

a. Diversion of Runoff from the RPB Basin to the Water Catchment Area. This alternative was developed to address localized flooding problems in Royal Palm Beach (sub-basin 15), since a significant portion of the urban damages in the western basin may occur in this sub-basin. The general plan would be to construct a pump station at the eastern boundary of sub-basin 15, approximately two miles north of Okeechobee Blvd., to pump runoff into a leveed conveyance canal. The canal would extend one mile east, across the Fox property (sub-basin 17), and discharge into the southwest corner of the City of West Palm Beach's Water Catchment Area. The plan would require improvement of the existing canal system in Royal Palm Beach to deliver the water to the pump station, purchase of appropriate right-of-way across the Fox property, construction of a pump station, and negotiation with the City of West Palm Beach to obtain appropriate access to the water catchment area.

D. ENVIRONMENTAL CONSIDERATIONS

Management of flooding problems of the C-51 basin must also consider environmental impacts within the basin itself, the WCA's, coastal estuaries, and Everglades National Park. A number of environmental factors and impacts have been evaluated as part of this project. In general, these environmental considerations have been applied as constraints to restrict the size or operation of the various project components and management methods. Each of the management elements of the C-51 backpumping plan was examined to determine the impacts on environmental quality, and an attempt was made to define ranges of operation that would have minimal adverse impacts. The environmental considerations that were the basis of this analysis are described briefly below.

1. **New Pump Station at S-319.** Pumping of water into the WCA's could have adverse environmental impacts by potential degradation of water quality and alteration of the natural hydroperiod. Pumping of additional water during the wet season may not be a significant problem because this water would pass rapidly through the sytem. Pumping during the dry season, however, may prevent seasonal drying of the marshes. Additional water may also provide environmental benefits to the WCA's, under certain conditions, by preventing extreme drying out of the marshes and reducing the incidence of muck fires.

2. **Water Control Structure at SR7.** Allowing the maximum amount of basin runoff to flow from the western C-51 basin to the eastern C-51 basin through the structure (S-155A), especially during the dry season, would reduce any environmental impacts to the extent that it would reduce the need to backpump stormwater into the WCA's.

3. **Non-structural Methods.** Implementation of the various non-structural flood control methods mentioned previously in this report may have environmental benefits by preserving portions of the remaining wetlands within the backpumping basin, by restricting future land development for agricultural and urban uses, maintaining the integrity of shallow groundwater resources of the basin, and incorporation of best management water quality practices (BMP's) into new development plans.

4. **Operational Methods.** Modifications to increase the regulation schedules for the WCA's could provide some benefits to wetlands of the Everglades National Park by permitting storage of additional water in the WCA system. This additional storage may reduce the need to make unseasonal water releases to the ENP and may provide additional water that could be released to the Park during dry periods.

To meet this goal, the South Florida Water Management District, the USCOE, and the National Park Service should continue their cooperative efforts on the South Florida Water Supply Study.

E. SCENARIOS EVALUATED

The planning elements described above were combined to develop several alternative future scenarios to represent water conditions in the C-51 basin which may occur with orwithout action by the SFWMD.

1. **Plans Not Requiring Action by the SFWMD.** Potential flooding in the C-51 basin, if no flood control plan is implemented by the SFWMDor the USCOE, will be aggravated by continued land development. New building criteria will be needed for future development and will increase the costs of new construction. Flood insurance costs may increase and federally-backed mortgages and other support for existing construction may become more difficult to obtain if adequate flood protection is not available. Expected damages and flood insurance rates may increase substantially as development continues within the basin. The frequency and severity of damaging floods will probably increase, resulting in economic losses and hardships to residents.

Scenario 1 Existing Conditions. The purpose of this scenario was to illustrate the impacts that could occur in the C-51 basin if a storm were to hit the area within the near future. This scenario demonstrates the damages that could occur in the basin if the M-1 Project is not implemented and hence, by comparison with scenario 2, indicates the potential benefits of the M-1 drainage plan to future water management in the western C-51 basin. The results of this scenario were discussed in detail in Part I.

Scenario 2. Existing Conditions with the M-1 Project Completed. This Scenario represents the impacts that would occur in the basin after construction of the M-1 Project, but with no action by the SFWMD. The assumptions are the same as for the first scenario with the exception that the M-1 Project is in place.

Scenario 3. Existing Conditions without the RPB Basin. This scenario is based on the assumption that no action is taken by theSFWMD, but that the M-1 acreage plan has been completed and the the Village of Royal Palm Beach implements a plan to divert runoff from sub-basin 15 to the water catchment area. The other assumptions are the same as for the first scenario.

Impacts of the first three scenarios were evaluated in both the eastern and western C-51 basins. This analysis indicated that there were no significant differences among these three scenarios in the eastern basin. The existing conditions in the eastern basin were described in Part I.

2. **Backpumping Plans.** These scenarios are based on the assumption that the SFWMD and/or the USCOE construct backpumping facilities for the western C-51 basin. One implication of such a plan is that the canal stages in C-51 will be lowered significantly while the pump is in operation. This reduction in stage will mean that structures discharging into the canal from various sub-basins may be able to discharge more water into the canal than they are presently allowed. One such basin is the Royal Palm Beach Basin (sub-basin 15). The Amil gates that discharge from sub-basin 15 into C-51 would be capable of discharging more than their presently allocated runoff if C-51 were backpumped. A scenario was considered to determine whether it may be desirable to restrict the operation of such structures during critical periods, if such restrictions on outfalls would provide significant benefits to the remainder of the basin.

Analyses of backpumping plans assume that future land use conditions exist in the basin, S-319 (pump) and S-155A (divide structure) have been built, and appropriate improvements have been made to the C-51 channel.

<u>Scenario 4 Backpumping with Controls on RPB Outfall</u>. This scenario assumes that a backpumping station has been built and that operation of the RPB Amil gates is restricted so that runoff from this basin does not exceed the levels presently specified in their surface water management permit.

<u>Scenario 5 Backpumping Without Controls on RPB Outfall</u>. This scenario was similar to scenario 4 with the exception that the Royal Palm Beach Amil gates were allowed to operate at their full discharge capacity.

<u>Scenario 6 Backpumping Without Controls on RPB Outfall, M-1 Acreage Included in the C-51 Basin</u>. This scenario assumes that the backpumping plan is

implemented, but that the M-1 drainage plan has not been completed. Otherwise, the assumptions are the same as for scenario 5.

Impacts of the various backpumping scenarios are the same for the eastern basin. In all of these cases, a new water control structure, S-155A, is in operation at SR7. This structure is assumed to be closed during major storm events so that all runoff from the western basin is diverted toward the west. In practice, this structure could be left open during a storm event if conditions in the eastern basin would permit such operation. Analysis of all backpumping plans also assumed that all of the necessary changes to the cross-section of the eastern portion of C-51, as proposed by the USCOE, have been completed.

3. **Non-Structural Plan.** A total non-structural flood management plan for the C-51 basin would be extremely difficult to implement based on inadequate flood protection criteria, since a substantial amount of existing development has been completed. Some existing landowners may wish to investigate methods for floodproofing homes that lie below the projected flood water stages in identified flood-prone areas. A non-structural plan for the basin could be devised to protect future development, but some existing facilities will still be at risk during moderately severe storms.

The initial steps of a non-structural plan to protect future development have been taken within this study. Detailed surveys and topographic mapping of the western basin and selected portions of the eastern basin have been made. Flood stages were calculated for.various storm events. Land use patterns have been delineated within the basin, especially in those areas where land elevations are below the expected flood stage during a 1-in-100 year storm event (which is a significant portion of the basin). Large areas exist within the basin that, in their current use, could be left unprotected or that could tolerate moderate flooding (agricultural uses such as sugarcane, improved pasture, etc.). TheSFWMD, the state, or the federal government could elect to purchase these lands or appropriate easements within these areas to somewhat reduce the need for additional flood protection and/or create areas for impoundment of excess basin runoff. Remaining flood prone areas in the basin could then be subject to special water management permitting criteria by the SFWMD and to zoning, density, and building restrictions, by county ordinance, to ensure that future construction and land use practices would provide adequate flood protection. Appropriate elevations would be established for house pads, roads, and septic drain systems. Adequate provision would be made to protect utility services and provide for emergency evacuation.

Another non-structural method would be to modify the local government comprehensive plans so as to provide for rezoning of flood-prone areas to the lowest feasible density or to provide for transfer of development rights or other incentives to discourage future development in flood prone areas. The county should also provide for strict enforcement of zoning laws and construction codes in the western basin.

PART III
POTENTIAL IMPACTS OF ALTERNATIVES

A. COSTS

Cost of the three main structural components of the backpumping plan have been estimated based on current costs for land acquisition, construction, operation, and maintenance. The basic design criteria were similar in nature, although reduced in magnitude, to the criteria that were used by the USCOE (1972) in the Design Memorandum for the C-51 basin. The costs, as determined by the USCOE, were updated by application of a 9% per year inflation rate. The costs estimated on this basis were compared with original cost estimates generated by the SFWMD based on current material and construction costs. The updated costs estimated by the USCOE and the original costs estimated by the SFWMD were in agreement within 3%. The costs assume that the project facilities will have a life expectancy of 50 years. Cost Breakdowns for the project are presented in Table 4.

Table 4. Estimated Costs of The C-51 Backpumping Project

a.	S-319 Pump station	
	1. Machinery	$5,350,000
	2. Buildings and support facilities	$5,440,000
	3. Bridge from S-5A	$40,000
	Total Initial Cost	$10,830,000
	4. Annual O&M Costs	$180,000
	5. Annual Equipment Replacement Cost	$70,000
b.	S-155A Divide Structure	
	1. 1000 cfs structure	$480,000
c.	Canal Improvements	
	1. Land acquisition	$710,000
	2. Excavation	$3,250,000
	Total Initial Costs	$3,960,000
	GRAND TOTAL Capital Costs	**$15,270.000**
	GRAND TOTAL Annual Costs	**250,000**

B. HYDROLOGIC IMPACTS

A detailed analysis of hydrologic impacts, with and without backpumping, is presented in Appendix A of this report. The following section is a brief summary of the impacts of the various water management alternatives that were discussed in Part II.

1. Scenarios Not Requiring Action by the SFWMD.

Scenario 1. Existing Condition. Hydrologic impacts of scenario 1 were described in Part I, which considers the existing status of the C-51 basin. The remaining five scenarios were compared with the existing condition to determine the benefits that could be derived from the various management options.

Scenario 2. Existing Conditions, M-1 Project Completed. The major difference that occurred in this scenario was that the flood stage in the Village of Royal Palm Beach (sub-basin 15) was reduced from 19.35 to 18.60 ft NGVD for the 1-in-10 year storm, and from 20.10 to 19.20 ft NGVD for the 1-in-100 year storm. Flood stages in sub-basins 2, 3, 5, 6, and 7 were reduced slightly due to a decrease in the amount of backwater from C-51.

Scenario 3. Existing Conditions, RPB Sub-basin Excluded. Flood stages in sub-basins 2, 3, 5, and 6 were reduced slightly (0.4 to 0.7 ft) relative to scenario 1. Flood durations were reduced by approximately two days. Sub-basin 7 would no longer receive backwater flow from C-51. The effects in the eastern C-51 basin were minimal. An analysis of discharge from S-155 indicated that exclusion of sub-basin 15 from the basin would have no significant effect on the amount or duration of floodwater discharged from S-155.

2. Backpumping Scenarios.

Scenario 4. Backpumping with Controls on the RPB Outfall. The results of this scenario indicated that flow would not occur over the south bank of C-51 into sub-basins 2, 3, 5, and 6. The flood stage in Royal Palm Beach peaked at 18.25 and 18.90 ft NGVD during the 1-in-10 and 1-in-100 year storms. These stages were 0.3 to 0.35 ft less than peak stages in scenario 2. Duration of flooding was much shorter under the backpumping plan. The backwater profile for the 1-in-100 year storm is much higher than for the other storm events because the total runoff of 3,682 cfs from the western basin exceeds the pump capacity of

3,400 cfs. This excess inflow would be temporarily stored in the canal and would increase the canal stage until local inflows become less than 3,400 cfs. Peak flood stages for the 1-10 and 1-in-100 year storms, with backpumping, are shown in Figure 4.

Scenario 5. Backpumping with No Controls on RPB Outfall. Peak discharges from the Royal Palm Beach outfall, with no controls, were 930 cfs and 1,500 cfs for the 1-in-10 and 1-in-100 year storms, respectively. Total runoff from the western C-51 basin was 3,935 cfs for the 1-in-100 year storm. Since 3,935 cfs exceeds the pump capacity, S-319 would have to operate at its maximum capacity of 3,400 cfs for approximately 66 hours. The backwater stage at Royal Palm Beach would reach 17.8 ft NGVD during the 1-in-100 year storm. The peak flood stage would be reduced only 0.05 to 0.10 ft below the levels that would occur with scenario 4. However, these peak flood stages would decrease more rapidly after the storm had passed.

Scenario 6. Backpumping, No Controls on RPB Outfall, M-1 Basin Included. Results for this scenario indicated that no impact would occur to sub-basins other than Royal Palm Beach. The limited discharge capacity of the Amil gate at C-51 would result in maintenance of higher stages and longer duration of flooding in Royal Palm Beach. Total runoff from the western C-51 basin would be 4,000 cfs, which would cause higher stages in C-51 under a 1-in-100 year storm event. The major differences in flood stages in Royal Palm Beach, under scenarios 4, 5, and 6, are presented in Table 5.

Flood stages in Royal Palm Beach would be substantially reduced if the Royal Palm acreage were excluded from the basin (scenarios 4 and 5). By comparison, unrestricted operation of the Amil gate structure at Royal Palm Beach has very little impact on peak stages in this sub-basin. It did reduce the duration of flooding, however.

The impacts of allowing unrestricted flow from the Royal Palm Beach sub-basin was investigated because it was felt that such operation might significantly reduce the amount of flood damages in Royal Palm Beach. Results of this analysis indicated that there was no substantial reduction in flood damages , since such damages are more related to water levels than duration of ponding.

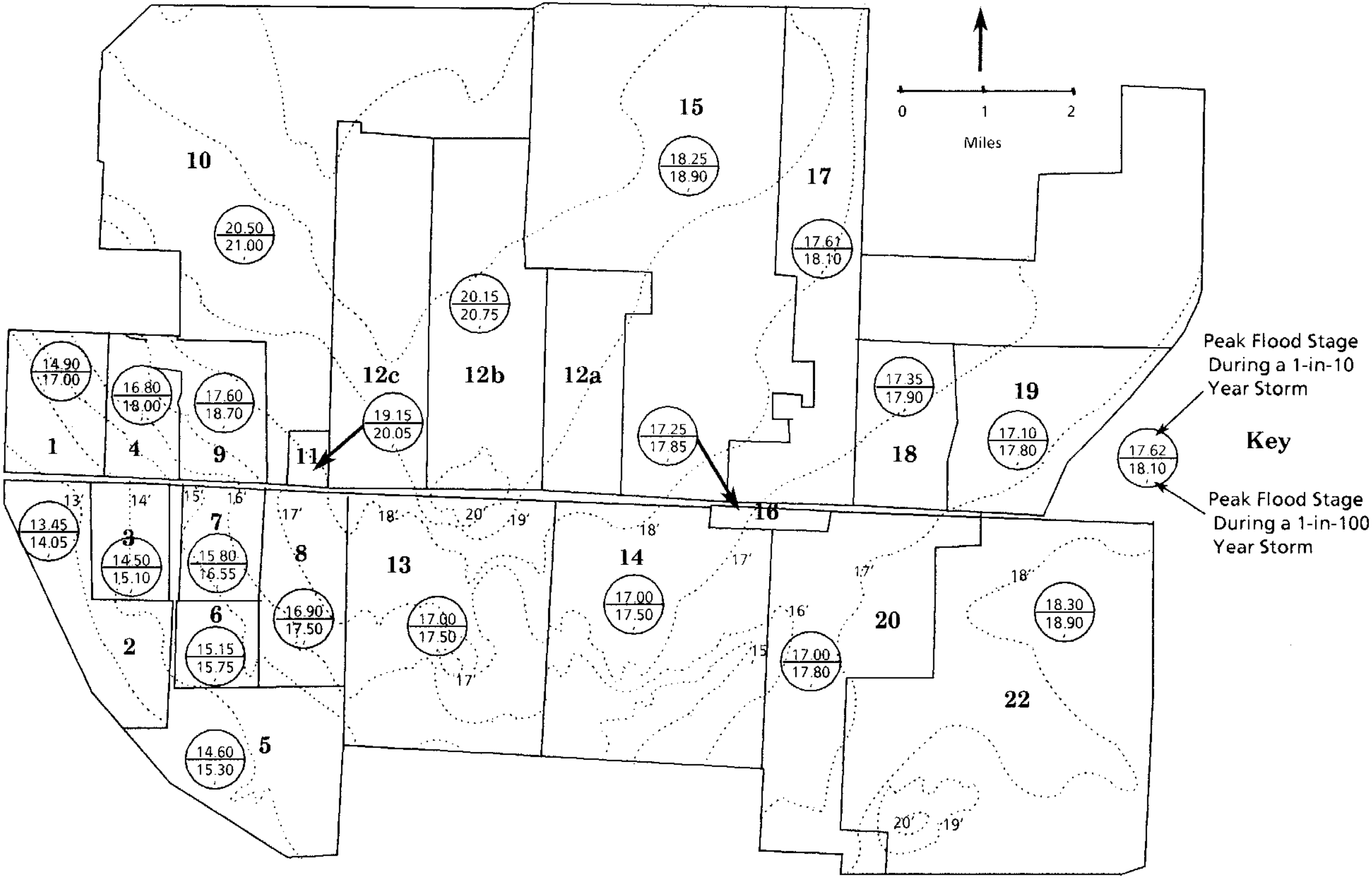

Figure 4. Peak Flood Stages for 1-in-10 and 1-in-100 Year Storm Events (ft NGVD) in the Western Subbasins of the C-51 Basin--With a Backpumping Project.

TABLE 5. Stages (Ft NGVD) in Royal Palm Beach with Backpumping Plans

	10-Year Storm			100-Year Storm		
Scenario No.	(4)	(5)	(6)	(4)	(5)	(6)
Day						
1	13.05	13.05	13.10	13.10	13.10	13.15
2	13.15	13.15	13.35	13.25	13.25	13.40
3	13.10	13.10	13.20	13.35	13.30	13.80
4	13.40	13.35	14.00	14.90	14.90	17.20
5	18.25	18.20	19.15	18.90	18.80	19.90
6	16.65	16.30	18.30	18.25	17.85	19.40
7	14.50	14.30	16.90	16.40	16.40	18.70
8	13.55	13.45	15.00	14.40	14.10	17.75
9	13.25	13.25	14.05	13.60	13.40	15.60
10	13.10	13.05	13.50	13.30	13.25	14.20

3. **Impacts in the Eastern Basin.** With the western portion of the C-51 basin backpumped, the peak discharges at S-155 would be 6,600, 7,155 and 7,481 cfs during the 1-in-10, 1-in-25, and 1-in-100 year storm events, respectively. The discharge hydrograph for S-155 indicated that runoff reached its peak in 10 to 12 hours after the most intensive rainfall (5.78 in/hr before noon of the 5th day) in the 1-in-100 year storm event. The discharge hydrograph under the backpumping scheme receded more rapidly than the hydrograph for the present condition without backpumping. The flood stage that occurs in C-51, with backpumping, was also considerably lower than the stage that occurs without backpumping. A comparison of the eastern basin flood stages with and without backpumping of the western basin are shown below.

Storm Return Frequency (Years)	Stage @ Summit Blvd. (Ft. NGVD) Without Backpumping	Stage @ Summit Blvd. (Ft. NGVD) With Backpumping	Stage@ SR 7 (Ft. NGVD) Without Backpumping	Stage@ SR 7 (Ft. NGVD) With Backpumping
10	12.25	10.94	17.96	14.08
25	12.70	11.45	18.10	14.40
100	13.00	11.80	18.26	14.58

4. **Effects of Various Structure Sizes for S-155A.** A range of structure sizes was considered for S-155A in an attempt to determine an optimum size for this

structure based on the amount of runoff generated by the basin and the frequency of operation of S-319. A series of computer runs were made, using the regional routing model, to determine the amount of water generated in the basin and the amount of water that would have to be backpumped to WCA-1 as a function of structure size. Structure discharge capacities ranged from 100 cfs up to approximately 1000 cfs. Detailed comparisons were made based on structure sizes of 300 cfs and 1000 cfs. Results of these analyses are summarized in Appendix C. Environmental and water quality impacts of S-155A were subsequently analyzed on the basis of structure capacities of 300 cfs and 1000 cfs. Results of these latter analyses indicated that it would generally be most desirable to pump the least amount of water into WCA-1; hence, the larger structure was clearly preferred over the smaller structure. The larger structure would also allow the flexibility to discharge amounts less than 1000 cfs to the eastern basin if desired.

5. **Royal Palm Beach Sub-basin Diverted to the Water Catchment Area**. The hydrologic analysis of this alternative considered four different pump sizes--250, 500, 750, and 1000 cfs--and determined the effects of each pumping capacity on expected flood stages in Royal Palm Beach with the 1-in-10 and 1-in-100 year storm events. Results of this analysis are summarized in Table 6. An elevation of 18.0 ft

Table 6. Flood Stages (ft NGVD) and Duration of Flooding for 1-in-100 yr and 1-in-10 yr Storm Events, Basin 15, Without the M-1 Acreage

Pump Station Size (cfs)	Number of Days Above 18.0 ft		Peak Stage	
	1-in 10yr	1-in100yr	1-in10yr	1-in-100yr
250	2.8	7.3	18.3	19.11
500	0.8	3.3	18.0	18.90
750	0	2.1	17.9	18.65
1000	0	1.2	17.1	18.39

NGVD was felt to represent a level of significant flooding. Days of flood duration above this stage are shown in the table.

During the 1-in-10 year storm event, significant flooding would occur with both the 250 and 500 cfs capacity pumps in operation. The 750 cfs pump would probably prevent significant flooding, since stages would reach 17.96 ft. The 1000 cfs pump would allow a peak stage of 17.11 ft and would thus eliminate the possibility of significant flooding in the 1-in-10 year event. Significant flooding would occur for all pump sizes up to 1000 cfs during the 1-in-100 year flood. Flood stages would peak at 19.11 ft and would persist above 18 ft for approximately 7 days with the 250 cfs pump. With the 1000 cfs pump, the maximum stage would be 18.39 ft and flooding would remain above 18 ft for 1.2 days.

The conclusion from this analysis was that the 1000 cfs capacity pump would provide an appropriate level of protection for the Royal Palm Beach basin. This is approximately the existing capacity of the Amil gate structure (under favorable tailwater conditions) that discharges into C-51 and is approximately 25% greater than the amount of runoff that is allowed by permit from Royal Palm Beach. Operation of the pump station for as much as five days, during a major storm event, would add approximately 9.3 inches of water throughout the water catchment area. This water could be removed from the catchment area after the storm had passed and other flood waters had been cleared from the basin.

C. FLOOD DAMAGE ASSESSMENT

The detailed hydrologic data for each sub-basin and for each scenario were used to estimate potential flood damages within the C-51 basin. A summary of expected flood damages for each scenario is presented in Table 7

Table 7. Preliminary Estimated Damages to Present Development Under Design Flooding Conditions in the C-51 Basin (Values in $1000's)

Scenario Number:	1	2	3	4	5	6
1-in-10 Year Flood						
Western Basin						
Agricultural	10,949	10,819	7,431	2,407	2,407	2,407
Urban	2,433	960	757	756	756	1,814
Eastern Basin						
Agricultural						
Urban[a]	13	13	13			
TOTAL	13,395	11,792	8,201	3,163	3,163	4,221
1-in-100 Year Flood						
Western Basin						
Agricultural	17,224	17,224	14,654	10,649	10,649	10,649
Urban	8,813	3,357	3,301	3,106	2,034	5,528
Eastern Basin						
Agricultural						
Urban[a]	111	111	111	0	0	0
TOTAL	26,148	20,692	17,066	12,755	12,683	16,177

[a]Lake Clarke Shores Only

Note: The damages computed in this table are only partial and do not represent a complete picture of flood damages in the basin. They are useful for assessing relative damage reduction capabilities of the scenarios.

D. WATER QUALITY IMPACTS

The primary water quality impacts of the various water management alternatives that have been proposed for the western C-51 basin are the potential effects of backpumping water from this basin into WCA-1. WCA-1 is a unique and irreplaceable natural resource and has been designated among the Outstanding Florida Waters by the Environmental Regulation Commission. Results of the water quality analyses that were conducted for this report are presented in Appendix B. The major points of the analyses that relate to backpumping are summarized in the following sections.

1. **Water Quality at S-319.** The quality of water that would be pumped by S-319 during flood conditions was estimated on the basis of existing water quality data for C-51. This analysis indicated that this water would be hard, highly mineralized, and alkaline; have an estimated specific conductance of 890 micromhos/cm, calcium and magnesium levels of 87.6 and 10.7 mg/L, respectively; and an average alkalinity of approximately 3.8 meq/L. The physical properties should include high color (143 Pt units) and low turbidity (7 NTU's). Total phosphorus should average 0.14 mg/L with approximately 70% (0.1 mg/L) as ortho phosphorus. Total nitrogen concentration should average 2.7 mg/L with approximately 35% (0.9 mg/L) as dissolved inorganic forms.

2. **Comparison of Water Quality from S-319 with Water Quality from Other Inflows to WCA-1.** S-319 water quality would have lower levels of specific conductance, organic nitrogen, total nitrogen, silicate, sulfate, sodium, potassium, magnesium, chloride, alkalinity, hardness, and total organic carbon than the existing inflows to WCA-1 (excluding rainfall). The quality of water from S-319 would also rank second lowest in nitrite, ammonia, and calcium. The pH, color, nitrate, and dissolved oxygen levels at S-319 would fall in approximately the mid-range relative to the existing inflows. The estimated average concentrations of ortho and total phosphorus at S-319 would rank second highest after S-5A.

3. **Comparison With Ambient Water Quality in WCA-1.** The quality of water that would enter WCA-1 from S-319 was compared with ambient water quality in the WCA's. WCA-1 was divided into three zones based on water quality characteristics--the perimeter canal, the interior marsh, and a transition

zone. Water quality from S-319 was compared with water quality in each of these three zones and with an averaged area-wide estimate of water quality. Average total nitrogen concentrations for S-319 were below the levels in the three internal WCA-1 zones and the volumetric average. The pH values for S-319 were within one unit of pH levels in the three zones and the volumetric average. The dissolved oxygen concentration at S-319 should average 4.0 mg/L which is about the same as oxygen levels in the perimeter zone. Therefore, operation of the proposed S-319 pump station should not have significant adverse effects on levels of total nitrogen, pH, or oxygen (excluding biological or chemical oxygen demands) in WCA-1.

The estimated average levels of turbidity, orthophosphorus, total phosphorus, nitrate, ammonia, and calcium at S-319 were greater than existing levels of these parameters in WCA-1. Thus, operation of the proposed S-319 pump station may increase the levels of these parameters in WCA-1. Estimated concentrations of alkalinity, color, conductivity, and nitrite at S-319 were less than the concentrations in the perimeter zone but greater than levels in the transition zone, interior zone, and the area average. Therefore, S-319 would not result in increased levels of these parameters in the perimeter zone but may cause higher levels of alkalinity, conductivity, color, and nitrite in the transition zone and interior marsh. Estimated levels of magnesium, potassium, and sodium for S-319 were greater than the levels in the interior zone. Chloride concentrations were greater at S-319 than in the interior marsh or than the area average.

E. LOADING ANALYSIS

The areal impact on WCA-1 of operating a 3500 cfs pump station at S-319 will be a function of the frequency of pumping and the stage in WCA-1. Operation of S-319 could significantly increase the loadings of nutrients and water into the area. The average annual increases in loadings would be 3% increase in water, 5% increase in phosphorus, and 2% increase in nitrogen loadings with a 1000 cfs structure at S-155A. If the structure capacity at S-155A were 300 cfs, average annual loadings would be increased by 11% water, 17% phosphorus, and 8% nitrogen. Since the nature and extent of any adverse impacts that these, or pollutant loadings, may have on WCA-1 are poorly known, it is advised that these loadings be minimized, (to the extent possible), in the following ways:

1. Minimize discharge volumes
2. Minimize frequency of pumping events by using the larger S-155A structure.

3. Minimize backpumping during low stage conditions, such as water supply backpumping.

Assuming that S-319 will be operated infrequently and will pump relatively small volumes of water during most of the year, the operation of this station would probably not result in any significant water quality or environmental degradation to WCA-1.

F. ENVIRONMENTAL IMPACTS

1. **Effects of Management of the C&SF Project.** In recent years, two major changes have occurred in the way that the Lake Okeechobee-Water Conservation Area system is managed which have profoundly influenced stage and discharge conditions in these areas. These changes are as follows: 1) the Interim Action Plan for management of discharges into Lake Okeechobee and 2) the WCA-2A drawdown.

Under the Interim Action Plan, most of the runoff water from the Everglades Agricultural Area is no longer discharged to Lake Okeechobee. Instead, this water is diverted south through S-6, S-7, and S-8 into the WCA's. The drawdown of WCA-2A has prevented this area from retaining any significant amount of the additional water that has been added to the WCA system due to the Interim Action Plan. The result is that this additional water must be released to WCA-3A and to the Everglades National Park or released through the coastal structures to tidewater. A regular routing model was used to simulate hydrologic conditions in the WCA's under differing management schemes. The so-called "base run" conditions for the model applied the modified WCA-2A regulation schedule and the Interim Action Plan to the historical data to simulate water conditions that would have occurred in the WCA's if the existing set of operating criteria were in effect.

2. **Effects of Size of the Basin Divide Structure (S-155A) in C-51.** Two discharge capacities were evaluated for the eastward discharge structure that would be placed in the C-51 Basin--300 cfs and 1000 cfs. Construction of the 300 cfs capacity structure would mean that the basin would generate 112,577 AF of water on an average annual basis that would have to be backpumped through S-319 into WCA-1. Construction of the 1000 cfs structure would require that only about 32,502 AF of water would be backpumped annually, which is 3% of the average annual inflows to WCA-1.

3. Methods and Base Run Conditions. Environmental impacts of backpumping water from C-51 into WCA-1 were analyzed on the basis of stage hydrographs of water levels in the WCA's that were generated by the Routing Model (see Appendix C). However, since WCA-1 is controlled by a flood control regulation schedule, it is recognized that much of the backpumped water will not remain in WCA-1, but will be discharged through the WCA system. This analysis examines the fate of this water as it moves through the three WCA's, and the probable environmental impact upon each.

The analysis centers upon simulated and historical hydrographs for each WCA for the time period of 1963-1981. The conditions that were simulated by the model include the following: (a) the base run, which is a simulation of the historical hydrograph that includes the extra water added by the Interim Action Plan and the effects of the revised WCA-2A regulation schedule--these actions alone cause considerable deviation from historical stages; (b) backpumping with a 1000 cfs eastward discharge structure ; and (C) backpumping with a 300 cfs eastward discharge structure in the C-51 basin. The hydrographs produced for the two backpumping schemes were thus compared to each other, the base run, and the historical hydrographs.

4. Impacts on the Water Conservation Areas.

a. WCA-1. The simulations for WCA-1 indicated that only two backpumping problem periods occurred during the nineteen years examined. These periods occurred in 1966 and 1968, which were both very wet years with high summer rainfalls. Simulated stages with backpumping exceeded both the historical stage and the stage with the base run (Interim Action Plan simulation). However, these high stages occurred during the rainy months when Everglades flooding is generally acceptable. While flooding of this magnitude may adversely impact some forms of wildlife (depending upon antecedent conditions), the impacts of these high water conditions are neither irreversible nor long-term. For instance, the deer population may be reduced, or wading bird feeding may be curtailed, but both would happen for one season only.

In all years except 1971 and 1981, during some portion of the year, the historical stages actually exceeded all of the simulated stages (base run, C-51 with 300 cfs capacity at S-155A, and C-51 with 1000 cfs capacity at S-155A). Stages were exceeded for various reasons, due primarily to water management decisions made by the USCOE or the SFWMD that deviated from the regulation schedule. On the descending side of the schedule (January through April), the model was programmed to follow a 17.0 ft NGVD to 14.0 ft NGVD schedule. In reality, there were many years during which the decision was made to follow an alternative schedule of 17.0 to 15.0 ft NGVD. Years when high historical stages were attained (1968, 1969, 1979) above the maximum of the schedule (over 17.0) were also deliberate. The model handles the water inflexibly and meticulously, without consideration of optional water management decisions. Also, the model incorporated large regulatory discharges to tidewater via S-39. These discharges aided in reduction of stages in WCA-1, and also permitted less water to be discharged into WCA-2A and WCA-3.

b. WCA-2A. The simulated hydrographs indicate that higher stages are achieved in WCA-2A with the 300 cfs capacity structure at S-155A than with the 1,000 cfs capacity for eastward discharge at S-155A. With the 1,000 cfs structure at S-155A, stages were increased very slightly above the base run stages in WCA-2A. These stage increases that occur due to backpumping of the western C-51 basin with the 1000 cfs structure at S-155A are small, of short duration, and hence are insignificant relative to the base run.

Results of these analyses showed that water stages under the base run conditions were significantly different from the historical hydrographs. The historical hydrographs are consistently much higher than the simulations. However, the simulated future conditions would pass more water through WCA-2A than has passed through this area historically.

The major reason for these apparent discrepancies is the change in regulation schedule that has occurred in recent years in WCA-2A. From 1962 until November 1970, WCA-2A was operated on a 14.5 ft NGVD maximum to 12.0 ft NGVD minimum regulation schedule. From 1970 until 1980, the schedule was from 14.5 to 13.0 ft NGVD. In November 1980, the water management agencies reduced the water regulation schedule to a 12.5 ft NGVD maximum and 9.5 ft NGVD minimum. The model attempts to follow this reduced water regulation schedule. Thus, large amounts of water are discharged from WCA-2A by the model that were not discharged historically, but would be under the drawdown schedule now in effect.

From the viewpoint of hydroperiod, the simulated hydrographs indicate that backpumping would not create a problem in WCA-2A.

c. WCA-3A. The model simulations for WCA-3A show an improved condition over the historical case in many years. For instance, during the first three years of impoundment (1963-1965) WCA-3A did not reach regulation schedule due to below normal rainfall conditions. With the base run and backpumping, stages in WCA-3A would have been better than the historical situation.

The year 1966 is an example of a period when the historical stages were higher than the model simulations. Analysis of conditions for this year also indicate the effects of the lack of minimum deliveries to Everglades National Park (ENP). The ENP discharge regime was put into effect in 1970. The model simulations include the minimum discharges to the Park. The adverse high water period that occurred historically in 1966 thus would have had the peak stages reduced considerably; however, the duration of high water would have been somewhat prolonged with C-51 backpumping.

The simulated stage would have exceeded historical levels in 1967, but this was due to conditions of the base run (IAP) rather than to C-51 backpumping. During high water periods (e.g., 1968-1970), simulated stages were above the historical levels. This additional water would have resulted in deeper water and prolonged flooding in WCA-3A, but these effects were primarily due to the circumstances of the base run, rather than the effects of backpumping. Backpumping acted to aggravate these conditions somewhat. The adverse effects of the serious drought that occurred in 1971, and the generally low water conditions of 1972 through 1980, may have been somewhat alleviated in WCA-3A under the conditions of the simulated model runs, but this beneficial impact was also due to the base run and not to C-51 backpumping.

In summary, the amount of water added to WCA-3A from either of the C-51 backpumping schemes is not sufficient to have any significant adverse impacts upon the WCA-3A hydroperiod.

WATER MANAGEMENT PLANNING FOR THE WESTERN C-51 BASIN

March 1984

Appendix A
HYDROLOGIC ANALYSIS OF FLOODING IN THE C-51 BASIN

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Appendix A

HYDROLOGIC ANALYSIS OF FLOODING IN THE C-51 BASIN

Methods

A hydrometeorological approach was applied according to the following procedure: 1) First, design rainfall events of selected frequency were determined for the drainage basin; 2) Direct runoff was computed from these storms; 3) Unit hydrographs were calculated for each selected sub-basin; 4) Design runoff was determined from each sub-basin by the application of runoff data to the unit hydrograph of each sub-basin; 5) The outflow hydrograph for each sub-basin was routed to the main channel according to the limitations of the existing or permitted outlet structures; and 6) Design hydrographs of the sub-basins were combined and moved downstream to the outlet point of C-51 (i.e. S-155) by the process of flood routing. Computer models based on the above procedures were used to predict runoff hydrographs from each sub-basin, selected sites along the main channel, and the outlets of the basin.

A. Delineation of Sub-basins

The sub-basins of C-51 (Fig. A-1) were defined based on a survey of all existing inflow points along the canal and SFWMD permits within the basin. The existing secondary water control structures along the West Palm Beach Canal (C-51) are described in Table A-1.

B. Selection of Design Rainfall Frequency

The most severe floods in the area are often associated with storms, or sequences of storms, which produce widespread rainfall of several days or longer duration. The selection of maximum one-day rainfall for the 1-in-10-year, 1-in-25-year, and 1-in-100-year storm events was based on a SFWMD technical publication (MacVicar, 1981). This one-day maximum rainfall was distributed, as antecedent conditions, into a 3-day rainfall event according to the Management and Storage of Surface Waters Permit Information Manual, Part IV (SFWMD, 1983). For the 1-in-10-year event, the September 1941 storm at the Loxahatchee rainfall station was used as a pattern for daily distribution of rainfall. For the 1-in-100 year storm, daily rainfall distribution was based on United States Army Corps of Engineers data (USCOE, 1972). Table A-2 presents the selected rainfall data for the 1-in-10-year, 1-in-25-year, and 1-in-100-year storms.

C. Time Distribution

The time distributions of rainfall for three selected historical storm events, recorded hourly at Loxahatchee and at the Palm Beach International Airport, were compared with SCS Type I and Type II rainfall distributions (Figure A-2). The actual rainfall distribution in these events did not follow either Type I or Type II exactly; however, the Type II distribution provided an adequate fit for design rainfall in this study. Type II represents regions in which the high rates of runoff from small areas are usually generated from summer thunderstorms.

D. Development of Unit Hydrographs

After a study of several alternate procedures for computing synthetic unit hydrographs, the following five parameters for each sub- basin were chosen for the development of a 30-minute unit hydrograph.

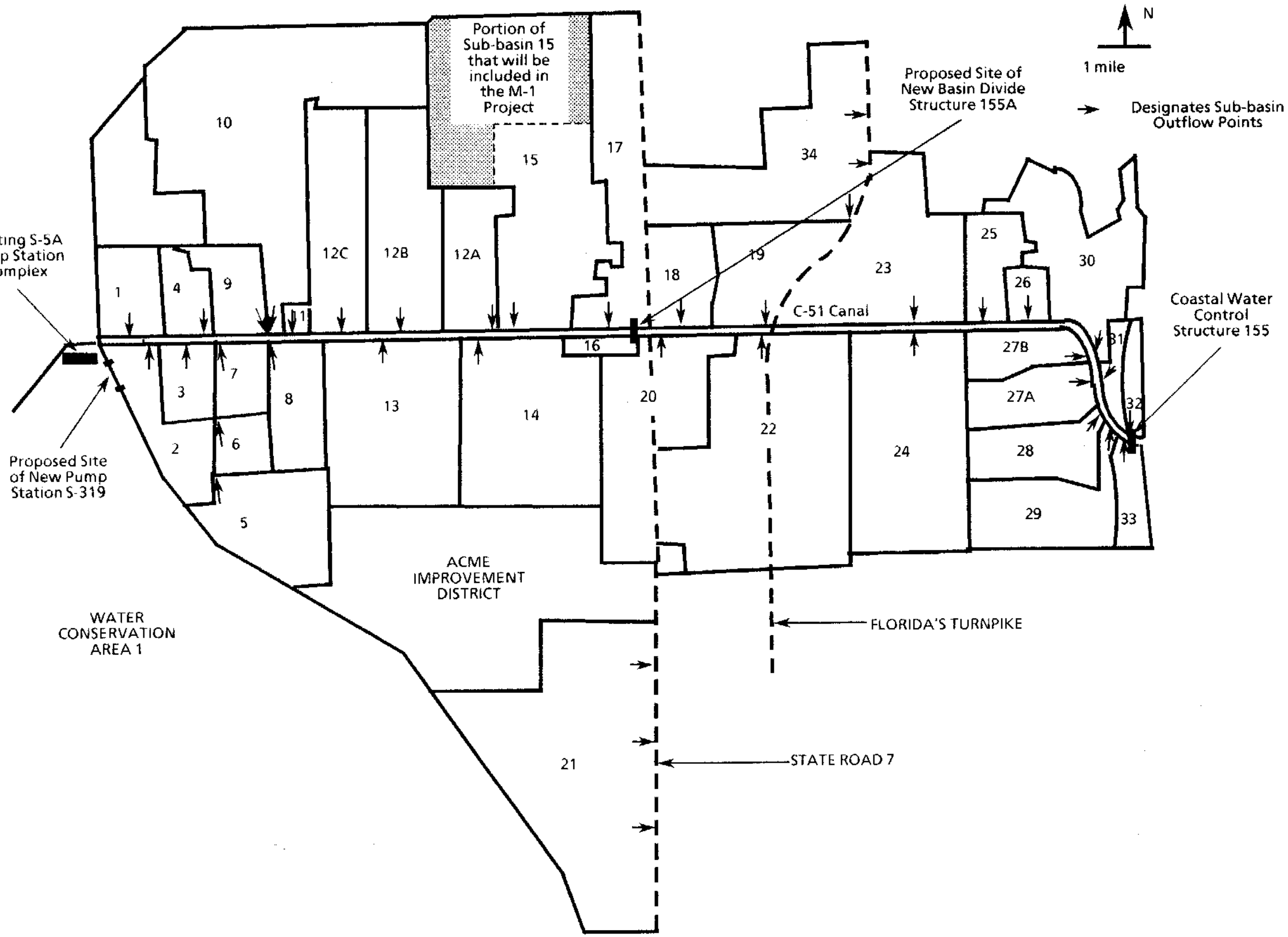


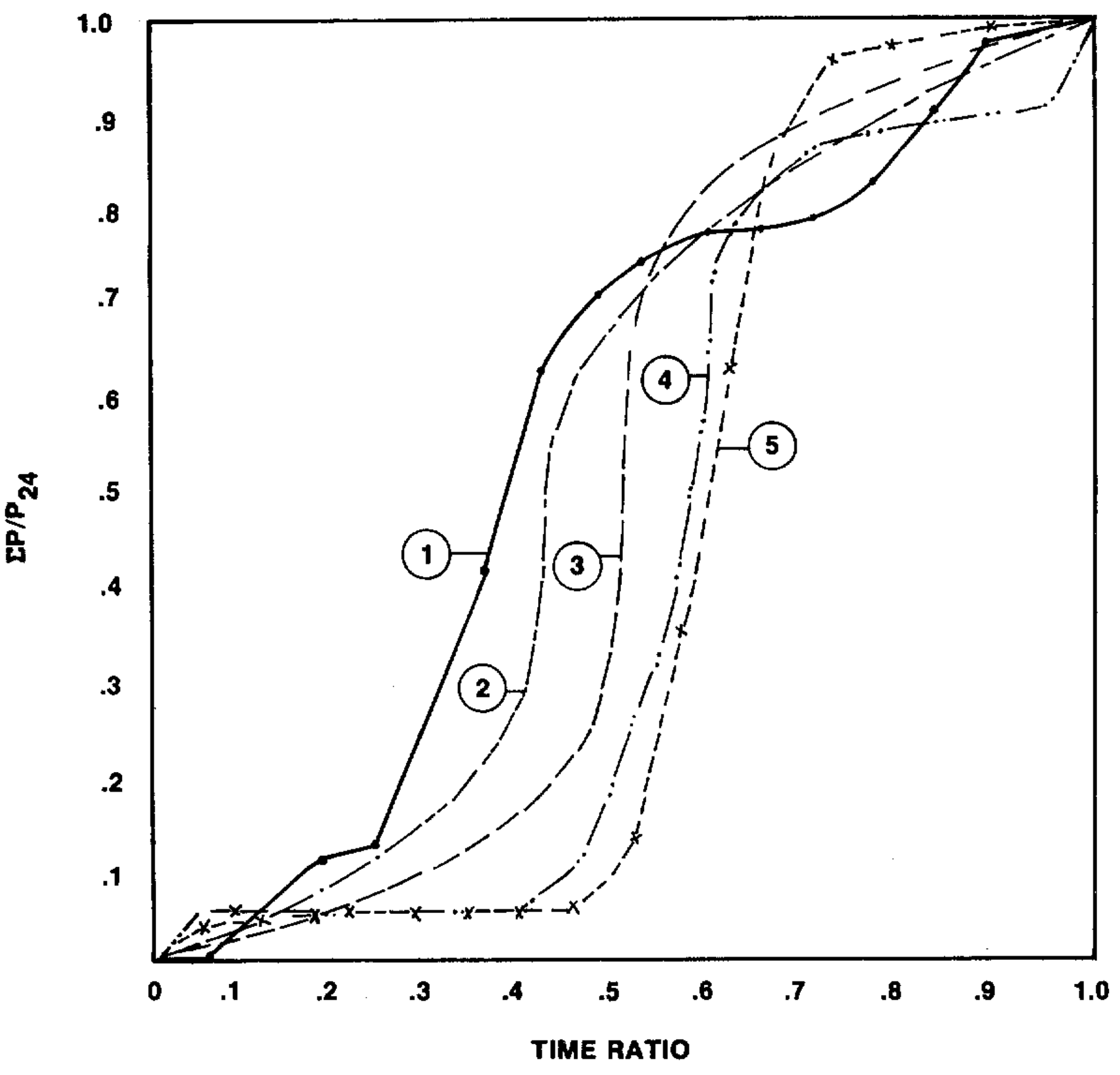
Figure A-1. Sub-basins of the C-51 Basin in Palm Beach County Florida and the Proposed Elements of the C-51 Backpumping Plan. Arrows Indicate Sub-basin Outflows

Appendix A

TABLE A-1. EXISTING STRUCTURES ALONG WEST PALM BEACH CANAL EAST OF S-5A(E)

Station	Reach No.	Sub-Basin	Type of Structure(s)
1090 + 47	1	32	1-60" storm sewer
1076 + 83	2	33	Open channel
1042 + 53	3	28	Open channel
		29	Open channel (LWDD E-4)
987 + 46	4	27A	Open channel
		31	3-48" x 475' RCP
969 + 00	5	27B	Open channel
		30	Open channel
890 + 00	6	26	Palm Beach AP pump
840 + 00	7	25	3-26" x 140' RCP
770 + 50	8	23	Open channel
		24	E-3 3 Amil-gates
620 + 00	9	19	Open channel
		22	E-2 gated structure
		34	
540 + 00	10	18	2 - 72" CMP
530 + 00	11	20	E-1 gated structure
		21	60" RCP
490 + 00	12	16	2 - 30" CMP
		17	3 - 72" CMP
420 + 00	13	15	2 Amil-gates (1 not fully operable)
382 + 00	14	12A	1 - 72" x 80' CMP
369 + 10	15	14	1 - 60,000 GPM pump
290 + 00	16	12B	3-bay, board-gated structure
261 + 30	17	13	1 - 60,000 GPM pump
213 + 71	18	12C	3 - 66" CMP
180 + 00	19	11	1 - 52" x 100' CMP
170 + 00	20	10	3 - 84" CMP
		9	1 - 54" CMP
160 + 00	21	8	1 - 18,000 GMP pump
		7	1 - 25,000 GMP pump
100 + 00	22	5	1 - 50,000 GMP pump through 1-84" CMP
		6	1 - 13,500 GPM pump through 1-48"x72' CMP
		4	12,000 GMP pump through 1-10'x10' box
60 + 00	23	3	1 - 25,000 GPM pump through 1-48"x60' CMP
40 + 00	24	2	1 - 34,000 GMP pump through 2-60" CMP
30 + 00	25	1	1 - 25,000 GPM pump with 1 - 72" CMP (8'x8' box under SR80)

Figure A-2 DIMENSIONLESS MASS CURVES OF RAINFALL FOR PALM BEACH COUNTY, COMPARED TO S C S CURVES



1 WPB AIRPORT - MAR. 28-29, 1982

2 SCS TYPE I

3 SCS TYPE II

4 LOXAHATCHEE - SEPT. 17-18, 1960

5 WPB AIRPORT - SEPT. 17-18, 1960

TABLE A-2. WEST PALM BEACH CANAL AREA DESIGN RAINFALL DATA (Inches)

DATE		STORM EVENT 10-Year	25-Year	100-Year
Sept.	1	0.45	0.50	0.50
	2	0.70	0.70	0.74
	3	1.24	1.53	1.97
	4	1.81	2.24	2.88
	5	8.50	10.50	13.50
	6	0.37	0.45	0.54
	7	0.34	0.38	0.53
	8	0.32	0.34	0.54
	9	0.31	0.32	0.51
	10	0.31	0.30	0.53
	11	0.31	0.31	0.50
	12	0.31	0.32	0.51
	13	0.31	0.32	0.50
	14	0.32	0.33	0.50
	15	0.32	0.34	0.49
	16	0.33	0.35	0.49
	17	0.33	0.34	0.48
	18	0.33	0.36	0.48
	19	0.34	0.36	0.48
	20	0.34	0.36	0.47
	21	0.34	0.36	0.46
	22	0.34	0.36	0.46
	23	0.34	0.36	0.46
	24	0.34	0.36	0.45
	25	0.34	0.36	0.45
	26	0.34	0.36	0.45
	27	0.34	0.36	0.45
	28	0.34	0.36	0.45
	29	0.34	0.35	0.45
	30	0.34	0.36	0.45
Total		20.99	24.26	31.67

1. Time of Rise, T_r. This parameter is defined as the time in minutes from the start of direct runoff to the time of peak runoff. A procedure developed by Tracor Inc. (1968) was adapted and adjusted with local data collected by the SFWMD.

For an "urbanized basin"

$T_r = 16.4 \phi L^{0.35} S^{-0.049} I^{-0.45}$ ....(1)

For a "rural basin"

$T_r = 3.4 L^{0.223} S^{-0.302}$ ....(2)

where:

L = length of the main channel (ft),
S = the slope of the main channel (ft/ft),
I = the percent of impervious cover for the sub-basin,
ϕ = an urbanization classification factor with a value of 0.6 to 1.2 (see Table A-3)

2. Peak Discharge, q_p. Runoff rates were estimated using the Cypress Creek formula (Stephens and Mills, 1968), which can be expressed as:

$Q = CM^{5/6}$ ....(3)

where:

Q = rate of flow (cfs) in 24 hour period,
C = a coefficient based primarily on the level of protection needed,
M = drainage area in square miles,

when the discharge (Q_{50}) is 50% of the peak discharge.

$$W_{50} = 2.91 \times 10^4 A^{0.959} q_p^{-0.983} \text{..................(8)}$$

5. Base Time, T_b. Tb is defined as the time in minutes from the beginning to the end of surface runoff for a given storm event. The unit hydrograph ordinates prior to the point of inflection can be determined from q_p, T_r, W_{75}, and W_{50}, as presented previously. The remaining portion of the unit hydrograph was calculated based on the method used by the Hydrologic Engineering Center (HEC) of the Corps of Engineers. The operation of such a unit hydrograph has been set up as a subprogram, UGRAPH, in this basin model (see Supplement of this Appendix).

E. Soil Types

Table A-4 presents the physical characteristics of each sub-basin in the C-51 basin. The sub-basin delineation was based on existing inflow to C-51 , topographic mapping, and data from the SFWMD permit files. The hydrological soil group was based on the detailed soil survey of Palm Beach County (Soil Conservation Service, 1978).

Group A soils have high infiltration rates even when thoroughly wet, and consist chiefly of deep, well- to excessively-drained, sand or gravel. Group B soils have moderate infiltration rates when thoroughly wet. These soils have a moderate rate of transmission. Group C soils have slow infiltration rates and a slow rate of water transmission. Group D soils have a very slow infiltration rate when thoroughly wet and therefore have high runoff potential.

Table A-4 indicates that most of the soils in the sub-basin have a dual grouping; i.e., A/D or B/D. The first letter applies to the drained condition and the second letter applies to the undrained, natural condition. Some of these characteristics can be changed by construction of drainage systems. In the selection of a CN value for each land use type, the CN value under the B group was used in the computation of a composite CN value, thus allowing for estimation of the maximum potential moisture storage in the sub-basin. Table A-5 presents the general land use classification and the numerical values of the five parameters that were used in the calculations of unit hydrographs for each sub-basin.

F. Sub-basin Hydrographs

A unit hydrograph for each sub-basin was developed based on the above procedures. Composite hydrographs from each sub-basin for the design storms were computed in the usual manner, by multiplying the ordinates of the unit hydrograph of each sub-basin by successive runoff increments (hourly steps), and summing up the partial hydrographs. The composite hydrograph was then routed through the outlet structure of each sub-basin to determine the inflow flood hydrographs to the main channel--i.e., C-51. A mass balance was applied to each routing at the outlet structure of each sub-basin. A subprogram called INFLO was developed to perform this task.

G. Hydraulic Design Procedures

Hydrographs of the sub-basins were combined by the process of flood routing to determine the changes that occur to the design flood hydrograph due to natural storage in each reach of the canal system. Figure A-3 shows the designated reaches

TABLE A-4. PHYSICAL CHARACTERISTICS OF THE C-51 BASIN

A. Under Present Conditions

Sub-Division	D.A. (Acres)	L (Mi.)	Slope (Ft/Ft)	Impervious (%)	Hydrological Soil Group	S (Inches)
1	1274.6	1.70	0.0000833	0	A/D,B/D	3.33
2	1768.4	3.03	0.0000781	0	A/D	4.28
3	818.7	2.37	0.00008	0	A/D	5.38
4	846.3	1.70	0.0000833	0	A/D,A	4.29
5	2222.0	2.31	0.000082	0	A/D	3.33
6	654.9	1.42	0.000080	0	A,A/D	5.38
7	854.0	1.70	0.000083	0	B/D	5.38
8	1491.4	2.33	0.0000813	0.5	B/D	5.38
9	1158.5	2.52	0.00075	5.0	B/D	4.70
10	9223.0	5.87	0.000089	0.9	B/D	4.70
11	298.0	0.66	0.000214	0.0	B/D	4.28
12A	1472.0	2.94	0.0000806	8.9	B/D	4.29
12B	2880.0	3.66	0.0000777	8.9	B/D	4.29
12C	3648.0	3.66	0.0000777	8.9	B/D	4.29
13	4747.0	3.0	0.000126	11.2	B/D	3.89
14	4748.0	3.0	0.000126	35.8	B/D	3.00
15	9037.2	7.00	0.0000811	15.9	B/D	3.89
16	209.0	1.38	0.000286	20.0	B/D	4.28
17	1232.0	2.0	0.0000952	33.0	B/D	3.16
18	1463.4	2.56	0.0000741	22.1	B/D	2.50
19	2199.5	3.75	0.000101	30.2	B/D	2.50
20	4083.7	4.4	0.0000646	11.4	B/D	3.33
21	8718.5	5.0	0.0000284	5.0	B/D	4.29
22	7983.5	4.19	0.0000905	17.2	B/D	3.51
23	4235.8	3.00	0.000079	37.2	B	3.33
24	5214.6	4.03	0.0000939	38.1	A/D	2.50
25	1280.6	2.37	0.000080	39.8	A/D,C	3.00
26	436.2	1.00	0.000588	80.0	A/D,B	1.11
27A	1298.6	2.94	0.000097	30.0	A/D	2.20
27B	1298.6	2.31	0.000082	30.0	A/D	2.20
28	1713.9	3.52	0.000081	38.6	A/D,C	1.76
29	2385.5	3.70	0.0000769	37.7	B,C	1.76
30	3019.8	4.36	0.0000761	51.2	A/D	2.05
31	563.8	0.85	0.000222	18.4	A	3.33
32	472.2	2.10	0.00018	36.1	A	3.33
33	732.5	1.89	0.00015	40.2	A	2.50
34	4412.5	4.00	0.000952	3.9	B/D	6.39

B. Under Future Conditions

Sub-Division	D.A. (Acres)	L (Mi.)	Slope (Ft/Ft)	Impervious (%)	Hydrological Soil Group	S (Inches)
1	1274.6	1.70	0.0000833	0	A/D,B/D	3.33
2	1768.4	3.03	0.0000781	0	A/D	4.29
3	818.7	2.37	0.00008	0	A/D	5.38
4	846.3	1.70	0.0000833	0	A/D,A	4.29
5	2222.0	2.31	0.000082	0	A/D	3.33
6	654.9	1.42	0.000080	0	A,A/D	5.38
7	854.0	1.70	0.000083	0	B/D	5.38
8	1491.4	2.33	0.0000813	0.5	B/D	5.38
9	1158.5	2.52	0.00075	5.0	B/D	4.70
10	9223.0	5.87	0.000089	9.6	B/D	4.28
11	298.0	0.66	0.000214	20.0	B/D	4.28
12A	1472.0	2.94	0.0000806	8.9	B/D	4.29
12B	2880.0	3.66	0.0000777	8.9	B/D	4.29
12C	3648.0	3.66	0.0000777	8.9	B/D	4.29
13	4747.0	3.0	0.000126	11.2	B/D	3.89
14	4748.0	3.0	0.000126	35.8	B/D	3.00
15	4557.0	4.73	0.00006	35.7	B/D	3.33
16	209.0	1.38	0.000286	20.0	B/D	4.28
17	3638.0	5.0	0.0000952	24.6	B/D	4.28
18	1463.4	2.56	0.0000741	45.2	B/D	2.50
19	2199.5	3.75	0.000101	44.7	B/D	2.50
20	4083.7	4.4	0.0000646	20.3	B/D	3.33
21	8718.5	5.0	0.0000284	5.0	B/D	4.29
22	7983.5	4.19	0.0000905	27.8	B/D	3.51
23	4235.8	3.00	0.000079	42.5	B	3.33
24	5214.6	4.03	0.0000939	38.1	A/D	2.50
25	1280.6	2.37	0.000080	39.8	A/D,C	3.00
26	436.2	1.00	0.000588	80.0	A/D,B	1.11
27A	1298.6	2.94	0.000097	40.4	A/D	2.20
27B	1298.6	2.31	0.000082	40.4	A/D	2.20
28	1713.9	3.52	0.000081	41.9	A/D,C	1.76
29	2385.5	3.70	0.0000769	41.4	B,C	1.76
30	3019.8	4.36	0.0000761	51.2	A/D	2.05
31	563.8	0.85	0.000222	18.4	A	3.33
32	472.2	2.10	0.00018	36.1	A	3.33
33	732.5	1.89	0.00015	40.2	A	2.50
34	4412.5	4.00	0.000952	31.3	B/D	3.89

TABLE A-5. UNIT HYDROGRAPH PARAMETERS

Sub-Basin	Basin Type	Basin Area (Acres)	TR (Min.)	Qp (cfs)	W50 (Min.)	W75 (Min.)	CN
1	Rural	1274.6	441.8	47.8	1258.3	603.0	75
2	Rural	1768.4	512.1	63.0	1308.4	618.1	70
3	Rural	818.7	481.2	33.7	1161.6	568.6	65
4	Rural	846.3	441.8	34.2	1177.6	575.2	70
5	Rural	2222.0	475.1	75.8	1362.5	636.6	75
6	Rural	654.9	429.4	27.9	1124.5	556.1	65
7	Rural	854.0	441.8	34.8	1168.0	570.7	65
8	Rural	1491.4	477.2	55.4	1265.6	602.8	65
9	Rural	1158.5	497.2	44.7	1227.7	591.3	68
10	Rural	9223.0	571.2	224.2	1838.4	799.7	68
11	Rural	298.0	269.2	14.3	1013.0	519.2	70
12A	Rural	1472.0	503.7	54.3	1275.4	607.4	70
12B	Rural	2880.0	534.9	94.7	1404.0	648.6	70
12C	Rural	3648.0	534.9	114.6	1461.0	667.4	70
13	Semi-Urban	4747.0	509.4	138.8	1558.3	702.0	72
14	Urban	4748.0	184.4	137.7	1570.5	707.1	77
15	Semi-Urban*	9037.2	610.5	217.5	1857.2	807.8	72
	Urban*	4557.0	256.8	146.7	1554.9	701.6	70
16	Urban	209.0	387.2	10.7	964.0	502.0	70
17	Semi-Urban	1232.0	164.1	46.3	1253.4	601.5	76
	Semi-Urban*	3638.0	628.9	114.2	1460.2	667.1	70
18	Semi-Urban	1463.4	501.0	53.4	1291.6	614.7	80
19	Urban	2199.5	177.2	74.8	1369.0	639.6	80
20	Rural	4083.7	589.2	120.3	1551.3	702.8	75
21	Rural	8718.5	777.3	214.6	1817.9	793.0	70
22	Semi-Urban	7983.5	526.5	199.7	1793.9	785.7	74
23	Urban	4235.8	181.0	125.6	1540.3	697.3	75
24	Urban	5214.6	164.4	155.1	1636.7	732.4	80
25	Urban	1280.6	161.7	47.8	1262.8	604.9	77
26	Airport	436.2	65.2	19.3	1096.6	551.2	90
27A	Urban	1298.6	196.2	48.1	1273.0	609.1	82
27B	Urban	1298.6	181.9	48.1	1273.0	609.1	82
28	Urban	1713.9	188.4	60.4	1329.0	627.8	85
29	Urban	2385.5	161.7	79.5	1393.4	648.3	85
30	Urban	3019.8	179.2	96.3	1446.6	665.7	83
31	Urban	563.8	126.9	24.3	1119.6	556.8	75
32	Urban	472.2	129.8	21.0	1092.2	547.5	75
33	Urban	732.5	120.3	29.9	1168.9	574.3	80
34	Rural	4412.5	512.6	133.4	1508.7	684.0	61
	Rural *	4412.5	178.5	130.5	1541.8	697.0	72

*Future land use conditions.

of the canal and the points of local inflow from sub-basins.

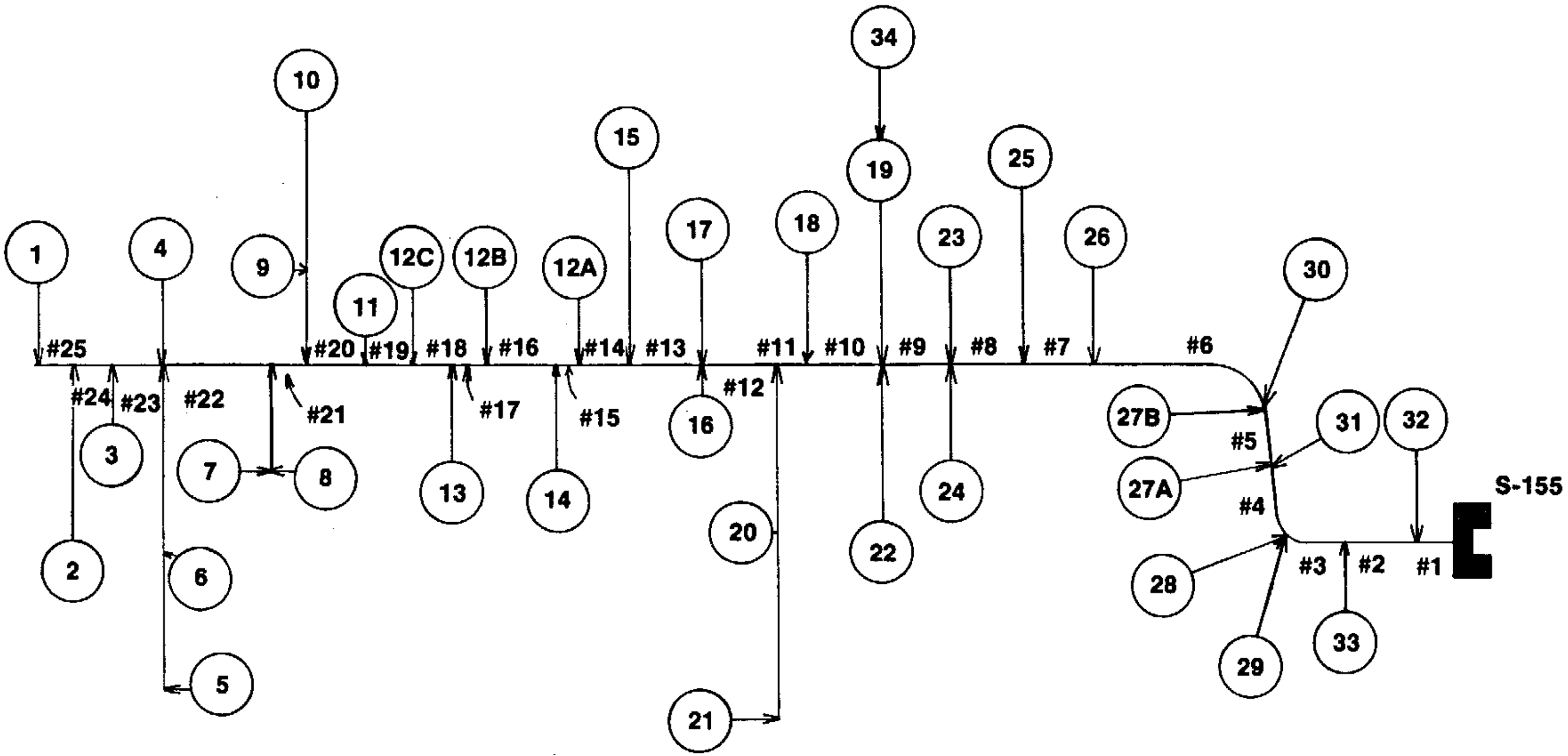
A modified Puls method was used in the hydrologic routing. Several backwater runs were first made using the HEC-2 program to establish the relationship between discharge and storage at each reach, relative to the operation of the outlet structure for the basin. Inflow hydrographs from the sub-basins were next added as local inflow to each reach and then the flood routing was calculated. This process was repeated for each reach until the outlet point of the entire basin was determined. The principle of mass balance was applied in this process to ensure that the total amount of sub-basin runoff was not changed.

H. Flood Duration Analysis

Flood duration analyses were required for flood damage assessments in each sub-basin. The computation of flood duration was based on the following steps:

1. Establish a stage-storage relationship for each sub-basin based on the latest available one foot contour data and permit information.

Figure A-3 LOCAL INFLOW POINTS FROM THE VARIOUS SUB BASINS OF THE C-51 BASIN THAT WERE USED FOR FLOOD ROUTING COMPUTATIONS



2. Compute total volume of water that enters the basin as rainfall.
3. Compute volume of runoff remaining in each sub-basin at the end of each day by subtrcting basin outflow from the net rainfall input. The INFLO computer model provided the following outputs for analysis:
   a. Rainfall input in inches to each sub-basin at each hourly time step.
   b. Rainfall excess (runoff generated) in inches to each sub-basin at each hourly time step.
   c. Average runoff per day in cfs from each sub-basin.
   d. The maximum peak flow in cfs during each day for each sub-basin and channel reach.
   e. Hourly and daily discharge for each reach of the main canal.

Under normal conditions, the flood duration analysis in each sub-basin was calculated directly as described above. However, overland flow occurred between several sub-basins in the western C-51 basin due to low land relief and a low crest elevation along the south bank of C-51. The elevation of the south bank varies from 15.4 to 17.0 ft NGVD in the reach from the station that is 2,100 ft east of S-5A(E) through the station that is 4,000 ft east of S5A(E). The peak stage for the 1-in-10 year storm exceeded 17.0 ft NGVD in this reach. Therefore, additional inflow would occur into the sub-basins on the south bank of C-51 from the remainder of the C-51 basin during these storm events.

The estimation of this additional inflow, due to backwater effects in C-51, was based upon the time history of backwater stages at State Road 7, and the total outflow from each sub-basin that was located west of State Road 7. During the 1-in-10 year event, the estimated flow contribution from the western basin to the east could vary between 500 and 1,000 cfs, depending upon the backwater stage at State Road 7. The remaining runoff that is generated in the western basin during this peak period would be stored temporarily in the lowland areas, such as sub-basins 1 through 7, and a small portion in the ACME Drainage District. Some drainage networks exist in the western basin. The available storage in these small open ditches was not included in the storage analysis; therefore, the computed flood stage may be lower than the actual stage.

I. <u>Design Assumptions</u>

The following operating and design assumptions were made in the computation of flows for the C-51 basin:

1. Lake Worth Road was used as a watershed boundary so that little or no flow out of Lake Osborne was assumed in this study.
2. It was assumed that Wellington's C-1 canal would be closed. A flow restriction to this structure, reducing the gate to a small culvert, would be required under the backpumping scheme.
3. Restrictions would be placed on gravity inflow structures under the backpumping scheme to insure equitable runoff allocations throughout the western basin.
4. Sub-basin 34, which is located north of Okeechobee Boulevard, west of the Florida Turnpike, and south of the water catchment area, has limited drainage capability under current conditions. This basin will have little or no drainage until a much later time, when tailwater conditions in the borrow canal on the western side of the Turnpike are at least 2 ft below ground level. Under back-pumping schemes, this area was allocated an inflow rate of 1-inch/24-hours.
5. The property that is bounded on the east by the west levee of the water catchment area, on the west by the Royal Palm Acreage and the Village of

Royal Palm Beach, on the north by the M canal and on the south by Okeechobee Boulevard will contribute little or no flow to C-51 due to lack of an outlet under Okeechobee Boulevard. Under backpumping schemes, this area was also allocated an inflow rate of 1-inch/24-hours.

The above assumptions will result in higher flows and increased duration of flooding in the recession portion of the hydrographs after the peak stages have been reached.

Calibration

Unit hydrographs were developed using data from a severe storm that occurred in southeastern Palm Beach County during April 24-25, 1979. Since no field data were available to prepare actual unit hydrographs for each sub-basin of the C-51 basin, there was concern for the validity of the theoretical procedures when applied to a flood analysis of the C-51 basin. The model was used to simulate several historical storms in the C-51basin . The results indicated that this model was adequate for flood analysis in this region. The simulations of these storm events are briefly described in the following paragraphs:

1. Storm of April 24-25, 1979. Rainfall over the basin during this storm ranged from 12 inches on the southeastern portion of the C-51 basin to 2.50 inches in the western basin (Fig. A-4). The peak flow that was generated by the model at the Palm Beach Locks was 2,605 cfs, as compared to the 2,620 cfs which was measured by the USGS during the same storm. The computed stage at Wellington Bridge, which is located about 12.4 miles upstream from the lock, was computed as 11.92 ft NGVD, as compared to the observed value of 11.70 ft NGVD.

2. Storm Events of September 17-18, 1960 and October 20, 1959. Hourly rainfall data were obtained from the Palm Beach International Airport and Loxahatchee weather station. The rainfall distribution of the storm event of September 17-18, 1960 was comparable to the Type II distribution (Figure A-5). The distribution for the storm event of October 20, 1959 was different from Types I or II (Figure A-6). Since the instantaneous discharge hydrographs for both events were available, the daily average flow was compared with computed values (see Fig. A-7). In general, the computed peak daily flows were within 5% of the recorded peak flows. For example, the daily recorded peak flows were 5,030 cfs for September 18, 1960, and 4,880 cfs for October 21, 1959, as compared to flows of 4,856 cfs and 4,980 cfs that were predicted by the model. The predicted time necessary to reach the peak was in close agreement with the observed time. However, predicted daily flow rates on the receding portion of the hydrographs were consistently lower than the observed values. These differences may have been caused by the following : 1) lack of adequate rainfall data over the basin (two stations were used); and 2) lack of land use, topographic, and channel cross-section data at the time of the event.

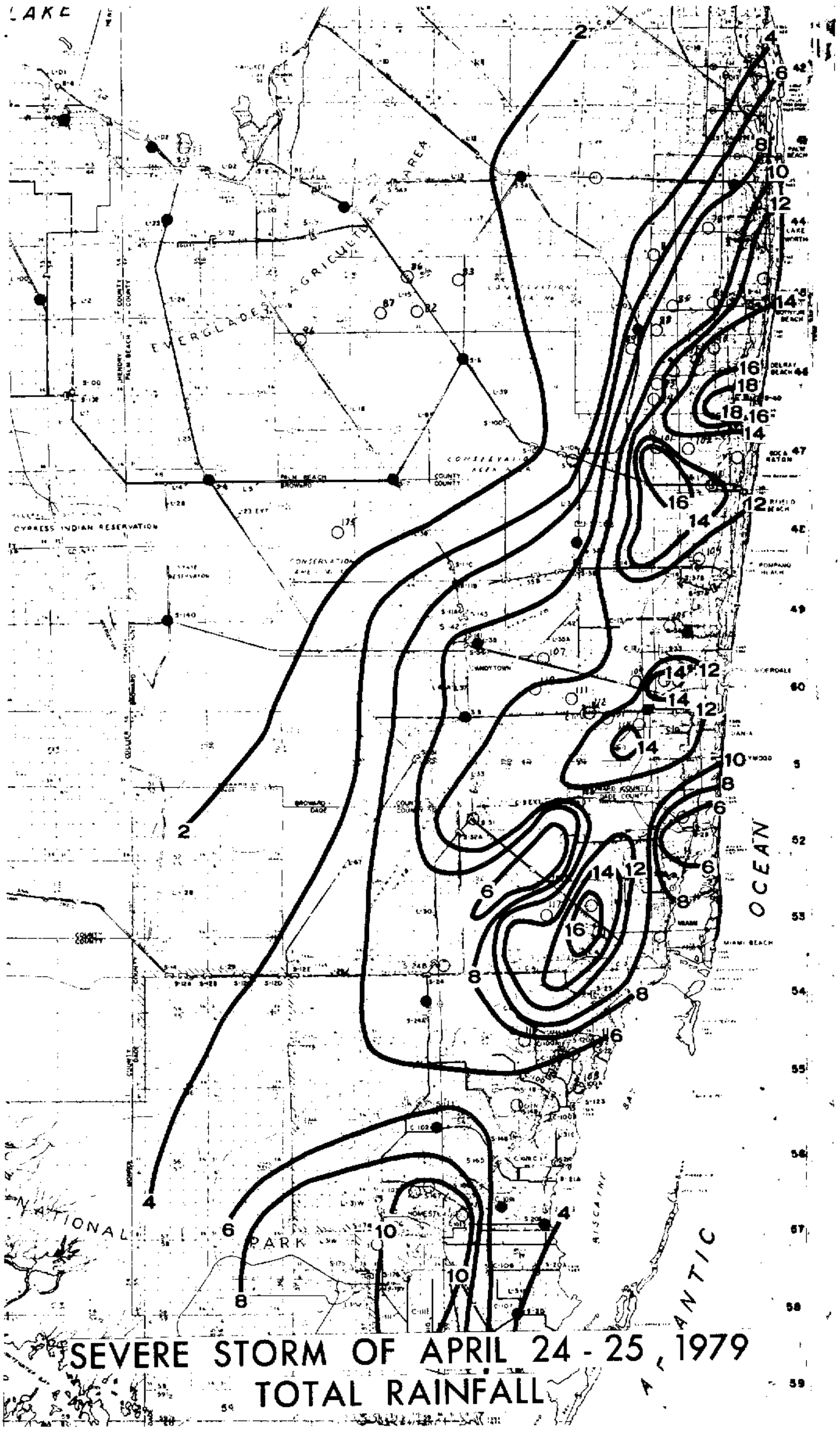
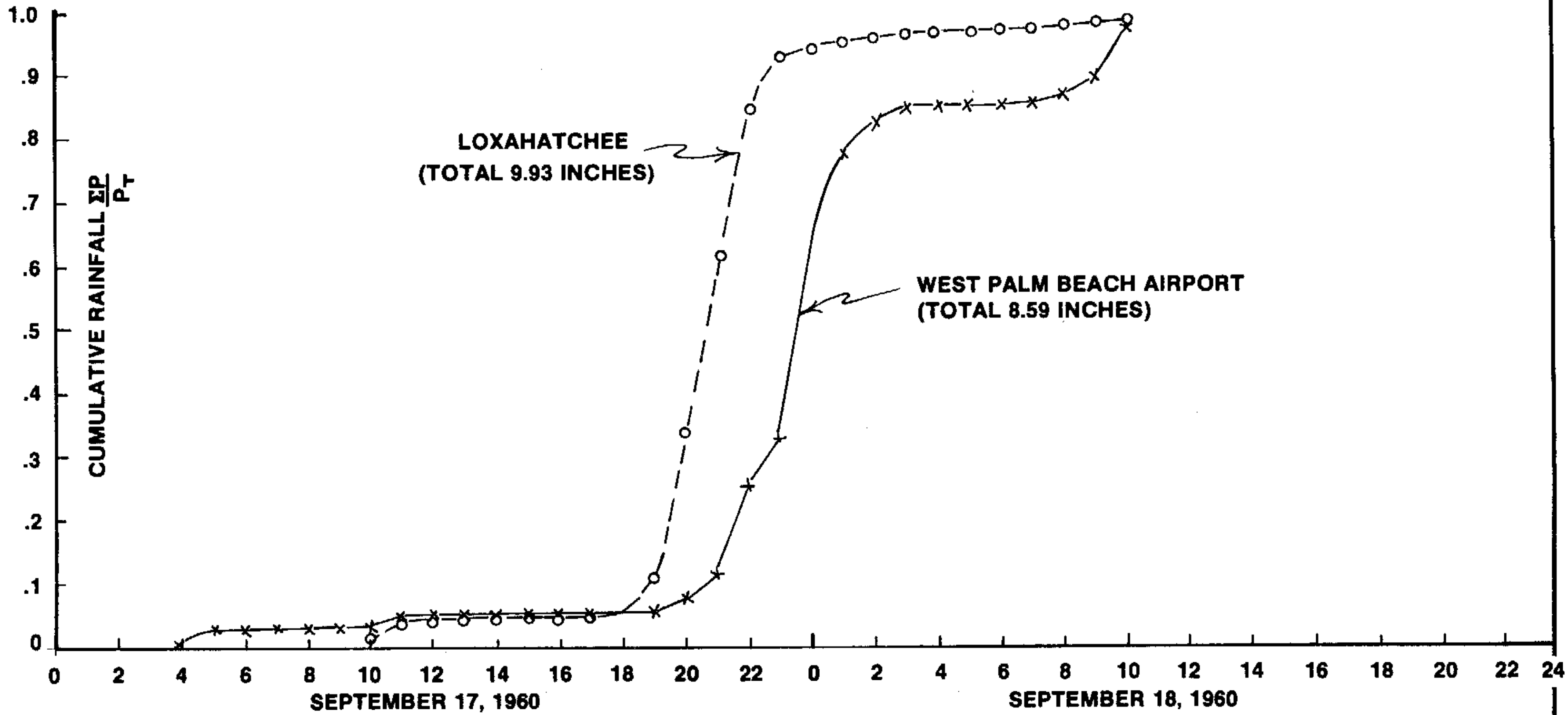

SEVERE STORM OF APRIL 24 - 25, 1979
TOTAL RAINFALL
EVERGLADES AGRICULTURAL AREA
OCEAN
ATLANTIC
NATIONAL
PARK

Figure A-4

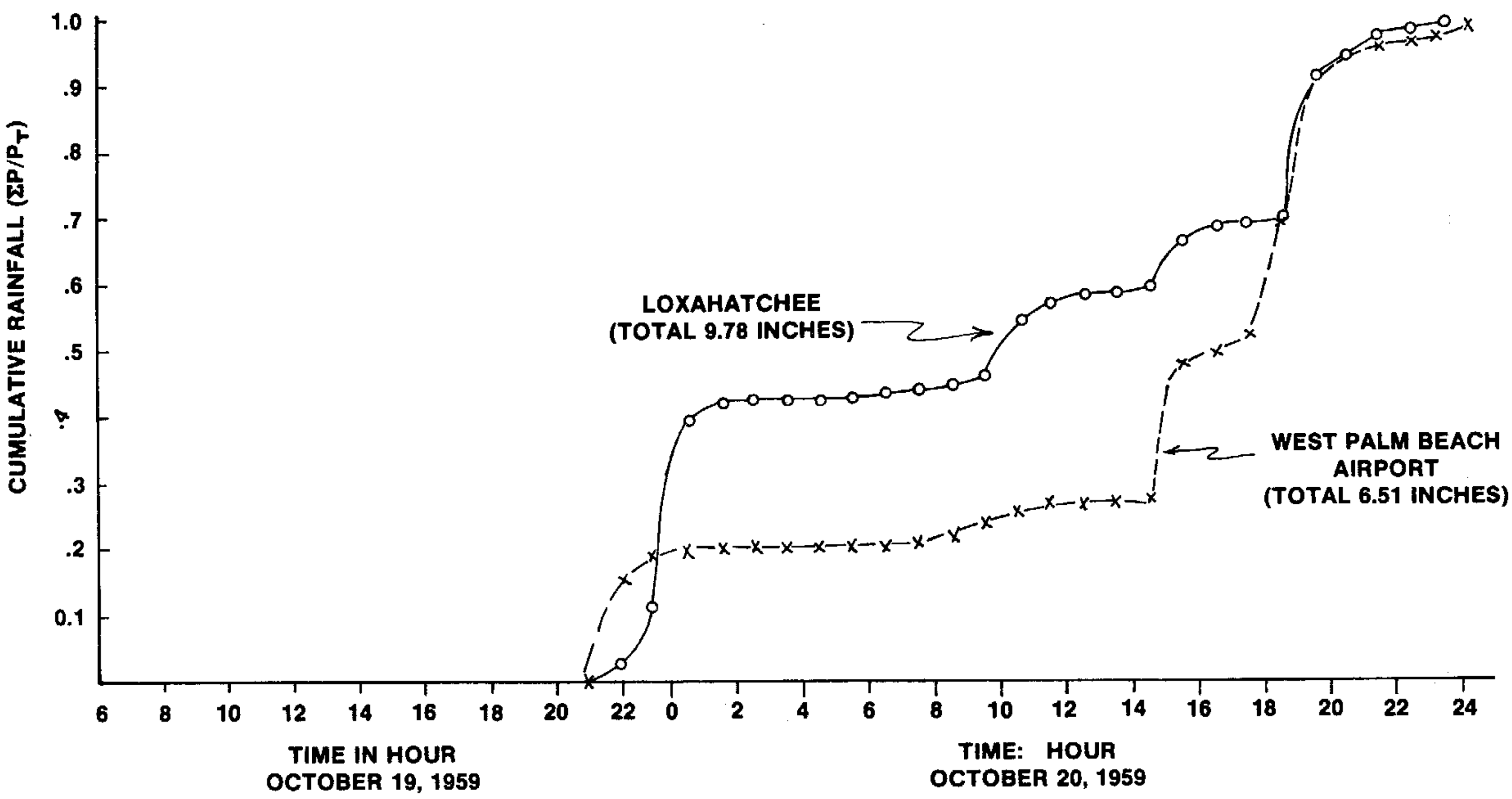
Figure A-5 CUMULATIVE RAINFALL DURING STORM EVENT OF SEPT. 17-18 1960 IN THE WEST PALM BEACH CANAL BASIN



BASIN: WEST PALM BEACH CANAL BASIN

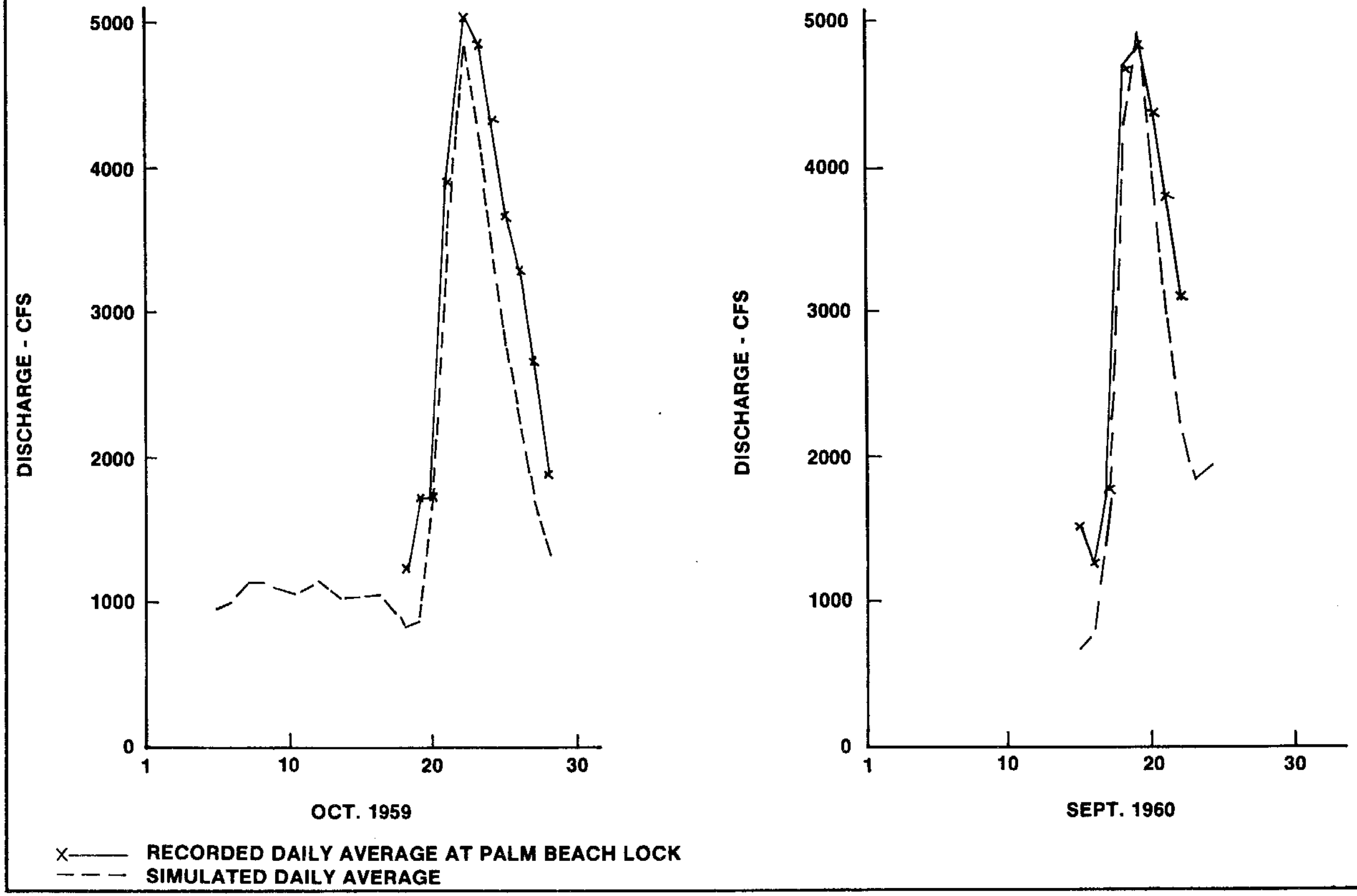
STORM: SEPTEMBER 17-18, 1960

Figure 6 CUMULATIVE RAINFALL DURING THIS STORM EVENT OF OCT. 19-20, 1959 IN THE WEST PALM BEACH CANAL BASIN



BASIN: WEST PALM BEACH CANAL BASIN
STORM: OCTOBER 19-20, 1959

Figure 7 COMPARISON OF PREDICTED AND RECORDED DAILY FLOW IN C-51 BASIN



Potential Impacts of Flood Events Under Existing Conditions

Rapid urbanization of the area during the past several years has increased concern that flooding may occur during severe storms in the western C- 51 basin (especially in the Village of Royal Palm Beach), and in the eastern basin in Lake Clarke Shores. Many homes in the C-51 Basin were constructed in the 1960's, prior to the District's regulatory program. House pad elevations that were used at that time were not based on the present knowledge of potential flooding. Back-pumping of excess runoff from the western C-51 basin to WCA-1 would provide some protection to existing properties from flood damage. Several case studies, both with and without backpumping, were evaluated in this report.

Land Use Data

The first step was to define the magnitude of the existing problem to determine whether a project was necessary or desirable. Existing land use was documented within the basin, both to provide an estimate of potential damages and to provide a data base for the modeling studies. Land use and land cover data are a prerequisite to the determination of runoff. "Existing land use" was defined from 1979-1980 aerial photographs and was supplemented with data from Palm Beach County building permit records. "Committed land use" refers to areas where no existing land use data were available, but where a development permit has been issued to a developer from a government entity such as the District. The permit information, as of February 1983, was used in this study. "Future land use" conditions that were used in the backpumping plans were based on the projections in the local government comprehensive plans, as supplemented by data from the SFWMD permit files

Rainfall and Runoff Conditions

The amounts of rainfall for the 1-in-10, 1-in-25, and 1-in-100 year storm events were compiled from a number of sources, but as much as possible, local rainfall patterns were used to distribute this rain. Flood stages peak at the end of the 5th day, which is the day of the heaviest rainfall. Over 50% of the total rainfall in that day occurs by 12 noon. With the wet antecedent soil conditions used in this study, all of this rainfall would become direct runoff and several inches of water would remain on top of the ground throughout the basin during the heaviest storm period. This overland flow eventually reaches its outlets at a later time. In Royal Palm Beach for example, the general land slope is toward the southeast and south. Therefore, most of the runoff generated in the M-1 acreage area accumulates in the lower land area of Royal Palm Beach.

The M-1 Project

Several rural residential projects are in various stages of development in the area north of western C-51. Two major projects, M-1 and M-2, are being constructed by the Indian Trail Water Control District. Both of these projects consist of 1.25 acre homesites. A 7-sq mi area (called the Royal Palm acreage or M-1 acreage) of the 28 sq mi area in the M-1 project, currently drains into C-51 via the main canal of Royal Palm Beach. This seven square miles will ultimately join the remainder of the M-1 area and drain north into the L- 8 canal upon completion of the M-1 project. The bonds for this project have been sold and construction of the drainage facilities is expected to be completed within two years. For the analysis of existing conditions, it was assumed that this project has not been completed. Additional scenarios were run to determine the impacts of completion of the M-1 Project.

Scenarios Evaluated Without Backpumping

A. Scenario 1. Existing and Committed Land Use Conditions with Royal Palm Acreage Area Included in the Royal Palm Beach sub-basin (no M-1 Project).

The Royal Palm Acreage will ultimately drain north to the L-8 canal. The purpose of this case study was to evaluate impacts to the Village of Royal Palm Beach and C-51 basin if a major storm occurs in the the basin before this project is completed.

Impacts in the Western Basin. Results of flood duration analyses for the various sub-basins in the western C-51 basin are presented in Tables A-6, A-7, and A-8 for design storms of 1-in-10, 1-in-25, and 1-in-100 years respectively. Flood stages at the end of the 5th day in Royal Palm Beach, for example, were estimated at 19.35, 19.70, and 20.10 ft NGVD for the 1-in-10, 1-in-25, and 1-in-100 year storms. Stages in C-51 during the 1-n-10 and 1-in-100 year events are presented in Figure A-8. Maximum stages in western C-51 occur in the reaches between SR7 and Royal Palm Beach, because sub-basins 2, 3, 5, and 6 become water storage areas that receive backwater from C-51. Sub-basins 7 and 8 and the ACME Drainage District would receive backwater from C-51 during the 1-in-100 year storm. Sub-basins 1, 4, and 9 would receive runoff from areas to the north such as Callery Judge Groves, Deer Run, Dellwood, etc., because the crest elevations of the divides between these sub-basins are low.

Impacts in the Eastern Basin. Figure A-8 also shows estimated water levels along the eastern reach of C-51 during 1-in-10 and 1-in-100 year storms. The peak stages reached 17.96 and 18.26 ft NGVD at SR7 during the 1-in-10 and 1-in-100 year storms respectively. The ground elevation at SR7 varies from 16 to 18 ft NGVD; therefore, the ground and roads in this area would be flooded. The flood stages at the Forest Hill Boulevard crossing of C-51 in the eastern C-51 basin would be 11.75 and 12.80 ft NGVD for the 1-in-10 and 1-in-100 year storms, respectively, and would be 12.25 and 13.30 ft NGVD at the Summit Boulevard bridge. House pads with elevations below elevation 13.50 ft NGVD in the area would probably be flooded during the 1-in-100 year storm.

B. Scenario 2. Existing and Committed Land Use Conditions Excluding the M-1 Acreage Area from The Village of Royal Palm Beach

The flood stages in this case are somewhere in between the stages that occur in scenario 1 and scenario 3. The routing results indicated that the major difference that occurred in this scenario was a reduction of the flood stage in the Village of Royal Palm Beach (Subbasin 15). The estimated flood stage dropped from 19.35 to 18.60 ft NGVD for the 1-in-10 year storm, and from 20.10 to 19.20 ft NGVD for the 1-in-100 year storm (See Figure A-9). Flood stages in sub-basins 2, 3, 5, 6, and 7 were reduced slightly due to a reduction of backwater inflow from C-51. Figures A-10 and A-11 indicate the duration of flooding in sub-basins 2 and 6 under various scenarios.

C. Scenario 3. Existing and Committed Land Use Conditions with the Village of Royal Palm Beach (sub-basin 15) excluded from the C-51 basin.

	Sub-basin Number:																				
Days	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	22
1	12.00	12.00	13.00	13.50	12.00	13.30	13.00	14.00	15.00	16.00	14.00	15.00	13.00	13.00	13.10	14.00	14.00	13.20	14.90	14.00	13.10
2	12.50	12.50	13.50	14.60	13.00	13,80	13.50	15.00	15.50	17.50	15.00	16.00	14.50	14.00	13.50	14.50	15.50	13.90	15.30	14.20	14.00
3	12.00	12.00	13.00	14.00	12.50	13.00	13.50	14.00	15.00	17.25	14.00	15.00	13.50	13.80	13.25	14.00	15.00	14.60	15.70	14.10	13.80
4	13.10	12.00	13.25	14.60	13.35	14.00	14.00	14.50	15.00	17.40	15.50	16.00	14.80	14.50	13.65	15.50	16.50	16.40	16.10	14.90	14.50
5	15.00	14.80	14.70	16.80	14.65	15.10	15.80	16.90	17.60	20.50	19.32	20.15	17.10	16.95	19.35	17.40	17.60	17.53	17.35	17.25	18.40
6	15.30	15.80	15.60	16.20	15.55	15.60	15.55	16.75	17.47	20.28	19.33	19.85	17.08	16.90	18.90	17.41	17.61	17.25	17.05	17.30	18.25
7	15.20	16.20	16.10	15.90	16.00	15.90	15.30	16.65	17.30	20.00	19.34	19.65	16.92	16.80	18.25	17.40	17.62	16.90	16.65	17.30	18.00
8	15.08	16.00	15.98	15.55	15.94	15.82	15.05	16.50	17.15	19.65	19.35	19.45	16.70	16.72	17.20	17.10	17.25	16.45	16.15	16.55	17.75
9	14.83	15.90	15.85	15.05	15.88	15.72	14.65	16.33	16.95	19.20	19.30	19.35	16.60	16.65	15.00	16.50	16.80	16.10	16.00	16.00	17.10
10	14.60	15.76	15.72	14.95	15.80	15.62	14.45	16.15	16.45	18.90	19.18	19.20	16.52	16.55	14.20	16.35	16.40	16.00		15.50	15.00
11	14.40	15.62	15.57	14.90	15.72	15.55	14.40	15.85	16.00	18.40	19.05	19.10	16.35	16.45	14.00	16.30	16.30			15.10	14.50
12	14.20	15.48	15.43	14.85	15.63	15.45	14.25	15.50	15.90	18.00	18.95	19.05	16.20	16.35		16.25	16.20			14.90	
13	14.00	15.34	15.30	14.80	15.55	15.35	14.10	15.35	15.80	17.50	18.88	18.90	16.05	16.20		16.20	16.10				
14	13.80	15.20	15.17	14.75	15.46	15.25		15.30	15.70	17.25	18.80	18.70	15.65	16.05		16.15	16.00				
15	13.60	15.06	15.00	14.70	15.37	15.15		15.25	15.60		18.65	18.50	15.25	15.95							
16	13.40	14.92	14.82	14.65	15.31	15.05			15.50		18.40	18.30									
17		14.78	14.65	14.62	15.23	14.95			15.40		18.20										
18		14.64	14.52	14.50	15.15	14.85			15.30		18.00										
19		14.50	14.39		15.05	14.75															
20		14.36	14.26		14.95	14.65															
21		14.22	14.13		14.90	14.55															
22		14.08	14.00		14.80																
23		13.94	13.87		14.70																
24		13.80	13.74		14.60																
25		13.66	13.61		14.50																
26		13.52	13.48																		
27																					
28																					
29																					
30																					

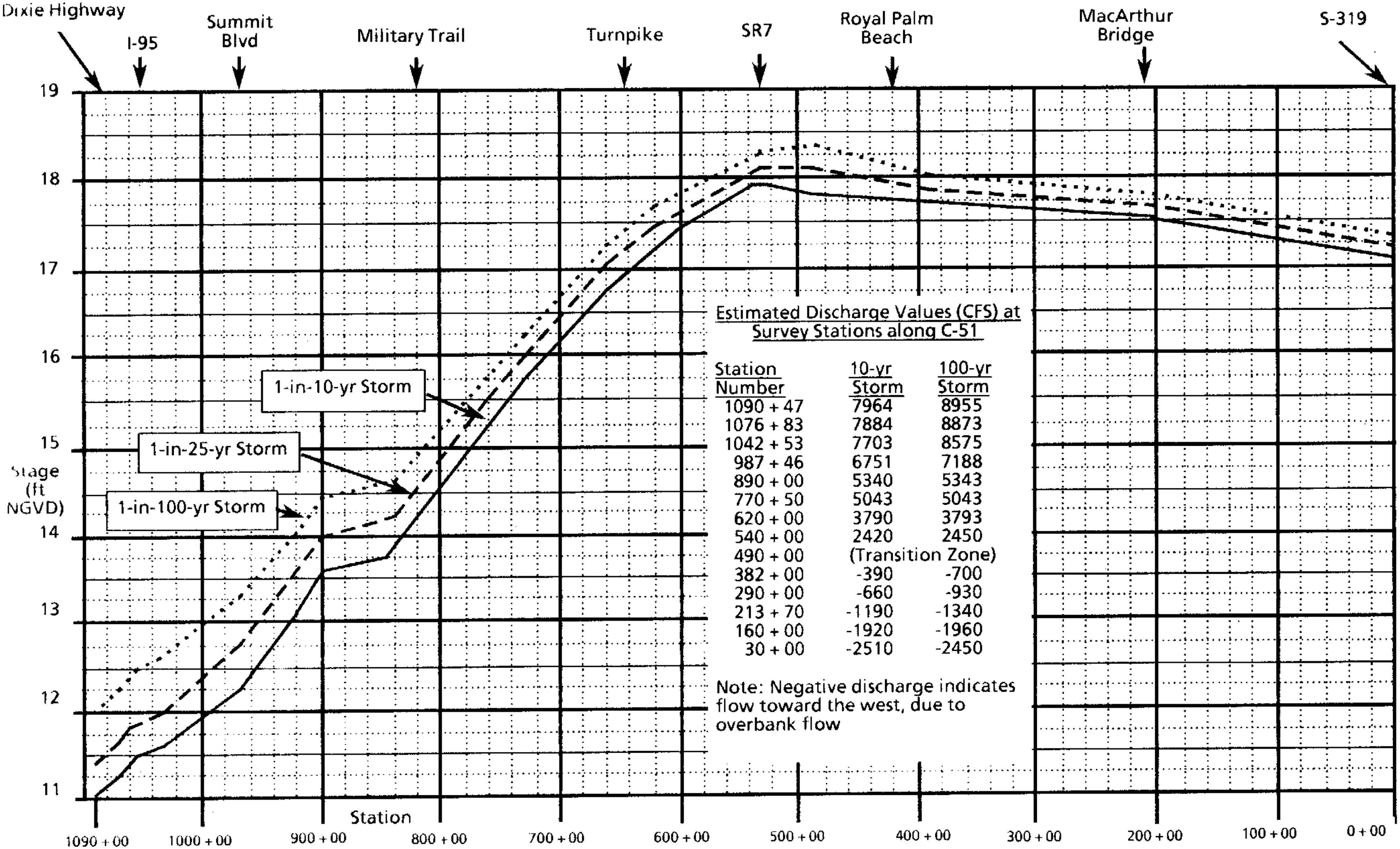
Table A-6. Flood Elevations (ft NGVD) and Durations for Sub-basins of the Western C-51 Basin During a 1-in-10 Yr Storm Event. Existing and Committed Land Uses with Acreage Area Included In Royal Palm Beach

	Sub-basin Number:																				
Days	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	22
1	12.00	12.00	13.00	13.20	12.00	13.30	13.00	13.50	15.00	17.80	15.00	17.50	13.00	12.60	13.20	15.00	14.50	13.50	15.00	14.10	13.20
2	12.50	12.50	13.50	14.50	13.00	13,00	13.50	13.50	15.50	18.25	16.00	18.20	14.50	12.70	13.60	15.50	15.50	14.00	15.50	14.25	14.20
3	12.85	12.15	14.00	14.00	13.50	13.35	14.00	14.00	16.10	18.10	16.80	18.15	14.40	12.80	13.60	15.50	16.00	15.00	16.00	14.50	14.60
4	13.25	13.00	14.55	14.70	13.60	14.00	14.50	15.00	16.25	18.40	17.20	18.40	15.30	14.40	15.10	16.00	16.30	16.00	16.25	15.50	15.60
5	16.40	14.90	14.90	16.75	14.83	15.25	16.05	17.25	18.00	20.70	19.65	20.40	17.40	17.30	19.70	17.65	17.80	17.75	17.45	17.60	18.70
6	16.50	16.10	16.00	16.60	16.00	15.48	16.00	17.15	17.90	20.50	19.65	20.20	17.30	17.22	19.35	17.66	17.82	17.55	17.30	17.60	18.60
7	16.65	16.75	16.65	16.35	16.65	16.50	15.85	17.05	17.75	20.25	19.66	19.90	17.25	17.15	18.95	17.70	17.85	17.45	16.85	17.65	18.45
8	16.48	17.10	17.00	16.15	17.00	16.90	15.50	16.95	17.60	20.00	19.67	19.70	17.12	17.07	18.20	17.66	17.80	17.00	16.40	17.35	18.30
9	16.35	17.10	17.00	15.95	16.98	16.90	15.25	16.84	17.45	19.65	19.70	19.52	17.00	17.00	17.15	17.45	17.40	16.55	16.00	16.60	18.10
10	16.20	16.96	16.87	15.50	16.91	16.82	14.70	16.70	17.30	19.35	19.70	19.40	16.90	16.95	15.15	17.10	17.25	16.00		16.15	17.75
11	16.05	16.82	16.74	15.40	16.83	16.74	14.65	16.55	17.15	18.90	19.60	19.30	16.78	16.85	14.15	16.55	16.70			15.70	16.80
12	15.90	16.68	16.61	15.20	16.75	16.66	14.60	16.38	17.00	18.45	19.55	19.22	16.65	16.80		16.50	16.50			15.25	15.00
13	15.75	16.54	16.48	14.90	16.67	16.58		16.18	16.85	18.10	19.45	19.10	16.52	16.70		16.40	16.50				
14	15.60	16.40	16.35	14.70	16.59	16.50		15.95	16.50	17.70	19.40	19.00	16.40	16.60			16.40				
15	15.45	16.26	16.22	14.40	16.51	16.42		15.65	16.00	17.65	19.33	18.85	16.25	16.50							
16	15.30	16.12	16.09		16.43	16.34		15.40	15.80	17.50	19.28	18.70									
17	15.15	16.00	15.96		16.35	16.26		15.25	15.60		19.22	18.50									
18	15.00	15.86	15.83		16.27	16.18					19.15	18.30									
19	14.85	15.72	15.70		16.18	16.10					19.10	18.10									
20		15.58	15.57		16.09	16.02					18.90										
21		15.44	15.44		16.00	15.94					18.80										
22						15.86					18.70										
23											18.60										
24											18.50										
25											18.40										
26											18.30										
27											18.20										
28																					
29																					
30																					

Table A-7. Flood Elevations (ft NGVD) and Durations for Sub-basins of the Western C-51 Basin During a 1-in-25 Yr Storm Event. Existing and committed Land Uses with Acreage area Included in Royal Palm Beach

	Sub-basin Number:																				
Days	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	22
1	12.00	12.00	13.00	14.00	13.00	13.00	13.00	14.00	15.00	17.00	15.00	17.50	12.50	12.50	13.20	14.00	15.50	13.50	13.00	13.50	13.50
2	12.50	12.10	13.40	14.00	13.30	13.50	13.50	14.50	15.70	17.50	16.00	18.20	13.50	12.75	13.60	15.00	16.00	14.00	14.00	14.00	14.50
3	13.00	12.20	13.30	14.50	13.40	13.50	13.40	15.00	15.85	18.30	16.50	18.25	15.05	13.00	14.10	15.20	16.20	15.00	14.50	15.00	15.00
4	13.50	13.30	13.55	15.10	13.85	14.20	14.50	15.20	16.35	19.05	17.50	18.90	15.95	15.40	17.55	16.15	16.70	16.10	15.50	16.10	16.00
5	16.50	15.30	15.15	18.00	15.15	15.75	16.55	17.50	18.70	21.00	20.05	21.00	17.52	17.50	20.10	18.00	18.05	18.00	17.85	18.10	19.00
6	18.20	16.30	16.20	18.30	16.20	16.20	16.40	17.40	18.70	20.50	20.05	20.65	17.30	17.35	19.80	18.05	18.10	17.88	17.65	18.25	18.95
7	18.16	17.00	16.82	18.30	16.82	16.80	16.25	17.32	18.50	20.25	20.05	20.40	17.00	17.15	19.50	18.06	18.10	17.70	17.40	18.25	18.87
8	18.10	17.60	17.50	18.25	17.50	17.40	16.10	17.50	18.25	20.05	20.06	20.22	17.00	17.05	19.10	18.07	18.10	17.50	17.05	18.25	18.80
9	17.90	17.65	17.60	18.18	17.60	17.50	17.35	17.60	18.10	19.65	20.07	19.95	17.00	17.05	18.55	18.10	18.10	17.25	16.50	18.00	18.70
10	17.80	17.65	17.60	18.10	17.30	17.60	17.60	17.50	17.95	19.10	20.08	19.77	17.00	17.00	17.65	17.85	17.75	17.00	16.20	17.45	18.60
11	17.70	17.50	17.47	18.00	17.00	17.30	17.48	17.43	17.88	18.95	20.10	19.65	16.85	16.85	16.15	17.57	17.35	16.60	16.00	16.75	18.45
12	17.55	17.35	17.34	17.72	16.95	17.00	17.36	17.36	17.73	18.00	20.08	19.55	16.75	16.75	15.00	17.30	17.20	16.00	15.70	16.25	18.10
13	17.35	17.20	17.21	17.65	16.88	16.93	17.24	17.26	17.60	17.50	20.06	19.50	16.65	16.65	14.25	16.95	16.70	15.70	15.50	15.85	17.60
14	17.20	17.05	17.08	17.55	16.80	16.85	17.12	17.16	17.45	17.00	20.05	19.44	16.50	16.50	14.15	16.80	16.60	15.50	15.40	15.60	16.50
15	17.05	16.90	16.95	17.40	16.75	16.78	17.00	17.06	17.30	16.50	20.00	19.35	16.35	16.35	14.00	16.50	16.50	15.30	15.30	15.50	16.00
16	16.90	16.75	16.82	17.30	16.69	16.70	16.88	16.95	17.15	16.00	19.95	19.29	16.25	16.25	14.00	16.40	16.40	15.10	15.20	15.40	15.75
17	16.75	16.65	16.69	17.15	16.63	16.62	16.76	16.84	17.00		19.90	19.23	16.05	16.05		16.25	16.30	14.90	15.10	15.30	15.50
18	16.65	16.51	16.56	17.00	16.57	16.54	16.64	16.70	16.85		19.85	19.18	16.00	15.90		16.00	16.20		15.00	15.20	
19	16.50	16.37	16.40	16.85	16.51	16.46	16.52	16.55	16.70		19.80	19.13	15.90	15.80		15.70	16.10				
20	16.40	16.23	16.30	16.70	16.45	16.38	16.40	16.38	16.55		19.70	19.05	15.80	15.70							
21	16.30	16.09	16.16	16.60	16.38	16.30	16.28	16.20	16.40		19.60	18.95	15.70	15.60							
22	16.20	15.95	16.03	16.47	16.31	16,22	16.16	15.92			19.50	18.80	15.60	15.50							
23	16.10	15.81	15.90	16.34	16.24	16.14	16.04	15.65			19.40	18.65									
24	16.00	15.67	15.76	16.21	16.17	16.06	15.92	15.40			19.30	18.50									
25	15.84	15.53	15.62	16.08	16.10	15.98	15.80	15.20			19.20										
26	15.70	15.39	15.48	15.95	16.03	15.90	15.68	15.00			19.00										
27	15.54	15.25	15.34	15.82																	
28	15.38	15.11	15.20	15.70																	
29	15.22	14.97	15.06	15.57																	
30																					

Table A-8. Flood Elevations (ft NGVD) and Durations for Sub-basins of the Western C-51 Basin During a 1-in-100 Yr Storm Event. Existing and Committed Land Uses with Acreage Area Included in Royal Palm Beach



Station Number	10-yr Storm	100-yr Storm
1090 + 47	7964	8955
1076 + 83	7884	8873
1042 + 53	7703	8575
987 + 46	6751	7188
890 + 00	5340	5343
770 + 50	5043	5043
620 + 00	3790	3793
540 + 00	2420	2450
490 + 00	(Transition Zone)	
382 + 00	-390	-700
290 + 00	-660	-930
213 + 70	-1190	-1340
160 + 00	-1920	-1960
30 + 00	-2510	-2450

Figure A-8. **Water Elevations (Ft NGVD) and Discharge Rates (CFS) in C-51 at Midnight on the 5th Day During 1-in-10 and 1-in-100 yr Storm Events. Existing and Committed Land Use Conditions, with Existing Canal Cross-Section and without Backpumping of the Western Basin. Canal Reach Numbers Correspond to Figure A-3.**

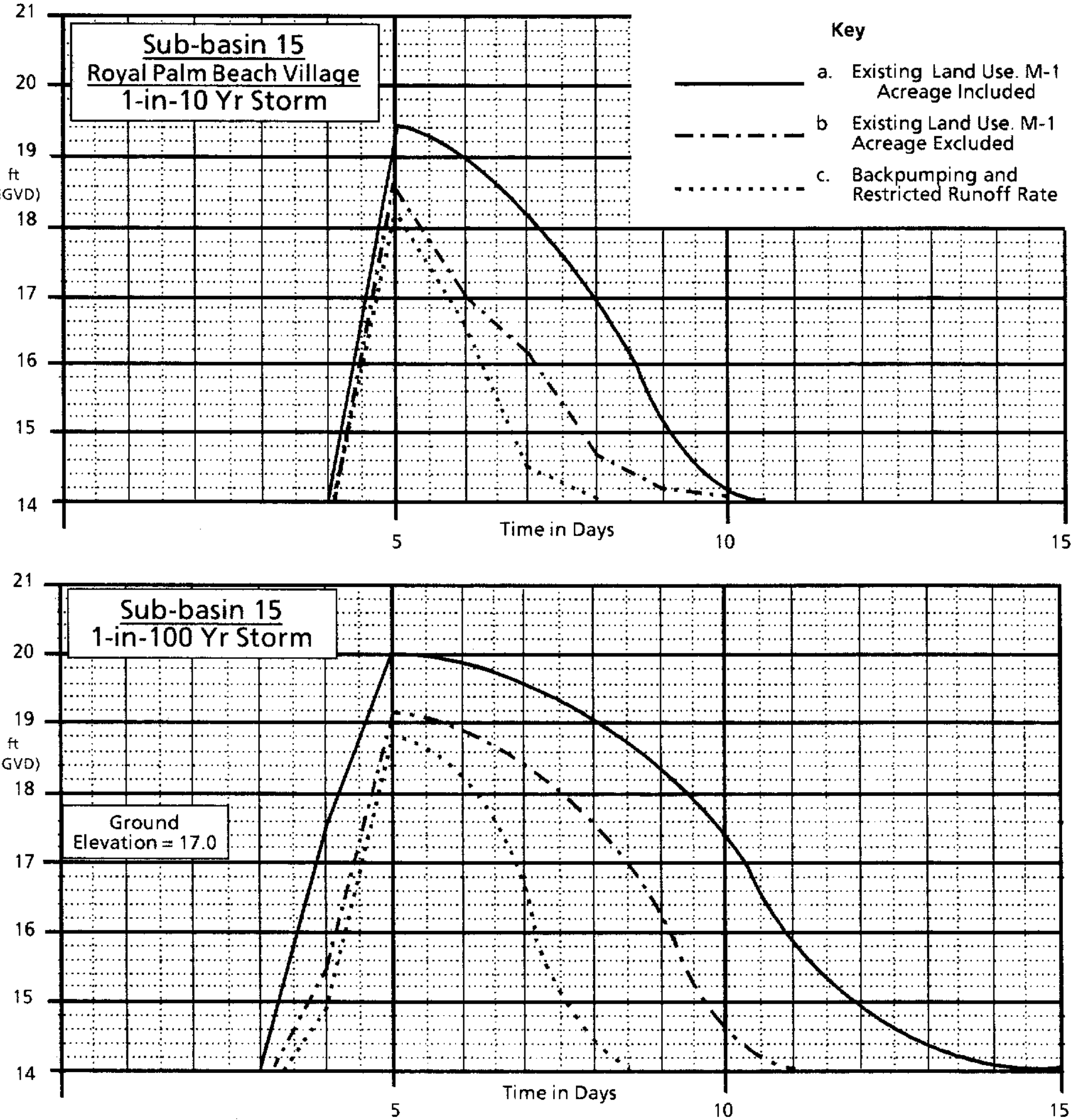


Figure A-9. Expected Stages for the 1-in-10 and 1-in-100 Yr Storm Events in the Royal Palm Beach Sub-basin (15) a. Existing Land Use, M-1 Acreage Included. b. Existing Land Use, M-1 Acreage Excluded. c. Backpumping, M-1 Acreage Excluded and Restrictions on RPB Outfall.

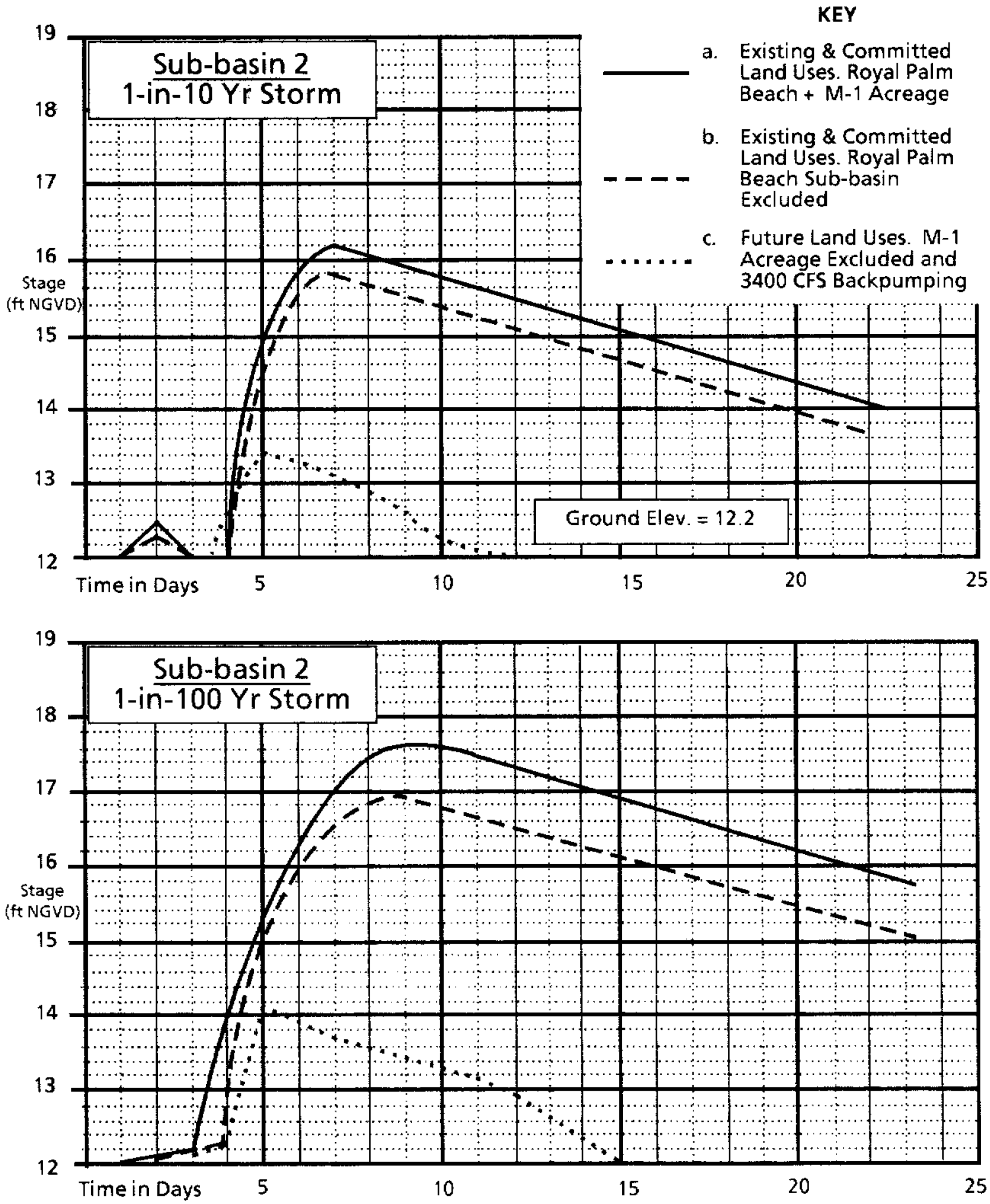


Figure A-10. Expected Stages for the 1-in10 and 1-in-100 Yr Storm Conditions in Sub-basin No. 2 (Sucrose Farms) of the C-51 Basin.

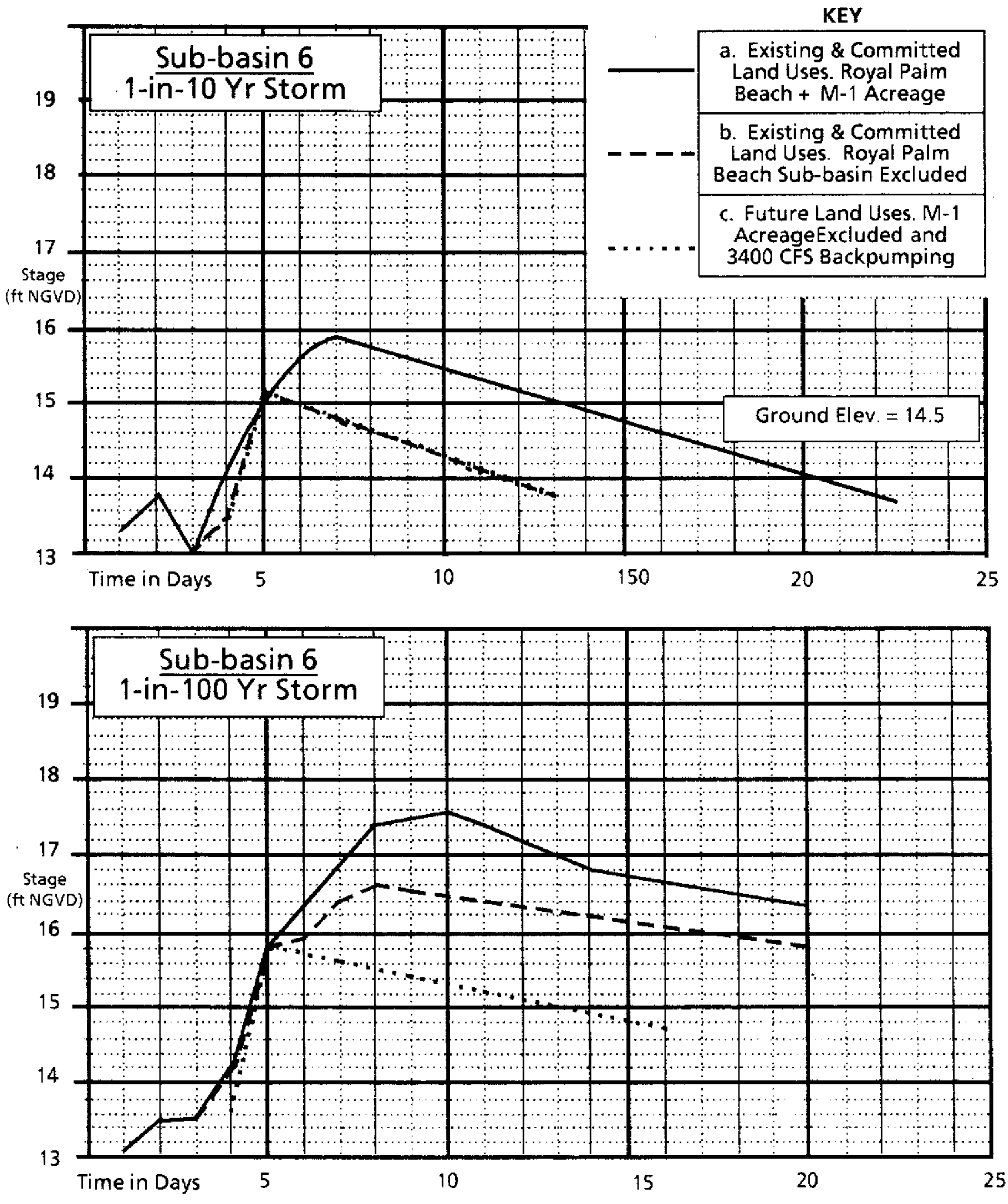


Figure A-11. Expected Stages for the 1-in-10 and 1-in-100 Yr Storm Conditions in Sub-basin No. 6 of the C-51 Basin.

The Royal Palm Beach sub-basin is a major source of storm water runoff to the western portion of C-51. If this stormwater runoff from Royal Palm Beach were diverted elsewhere, such as into the water catchment area, flood stages throughout the western C-51 basin may be reduced significantly. One alternative plan would be to pump the runoff from the Village of Royal Palm Beach into the water catchment area via an above-ground flowway.

Figures A-10 and A-11 show the duration of flooding that could occur in sub-basins 2 and 6 if runoff from the Royal Palm Beach sub-basin were removed from C-51. Tables A-9 and A-10 present the results of flooding analyses in the western sub-basins under the 1-in-10 and 1-in-100 year storm events for this scenario. Flood stages in sub-basins 2, 3, 5, and 6 were reduced slightly (0.4 to 0.7 ft) relative to scenario 1. Flood durations were reduced by approximately two days. Sub-basin 7 would no longer receive backwater flow from C-51. The effects in the eastern C-51 basin are minimal. Figure A-12 shows total discharge at S-155 during a 1-in-100 year storm for scenarios 1 and 3, and indicates that there is no significant effect in the eastern basin from removal of sub-basin 15.

<u>Potential Impacts of Backpumping</u>

Extensive flooding would occur in the western C-51 basin and portions of the eastern basin under existing conditions, with and without the M-1 Project. Backpumping would offer substantial relief from this flooding. Therefore three scenarios were evaluated that include construction of backpumping facilities. The backpumping plan would require construction of a new pump station, S-319, to pump water from the western basin into WCA-1. This station would be located at the western end of C-51 near S-5AE. An intermediate structure, S-155A, would be constructed west of SR7 to maintain optimum water levels in western C- 51 and allow flows to the east except during severe storm conditions. Enlargement of C-51 would be required in both the eastern and western basins to allow adequate transfer of water to S-319 and to S-155.

<u>Assumptions for Backpumping Plans</u>.

Future land use data were used in the evaluation of backpumping scenarios, since it may be several years before a backpumping plan could be implemented. The basin was divided into eastern and western basins at SR7, where the new structure, S-155A would be located. It was assumed that this structure would be closed whenever discharges from the western C-51 basin exceeded 1000 cfs.

Approximately 3,223 cfs of discharge would be generated from the western portion of the C-51 basin under a 1-in-10 year storm, which is about equal to the total amount of runoff that has been allocated by the District among surface water management permittees in this basin. Discharge records at the Palm Beach Locks indicate that the base flow from the C-51 basin is low (approx. 20 cfs). However, seepage along L-40 of WCA-1 is between 2 and 4 cfs/ft of head/mile . With the proposed channel improvements in western C-51, seepage during flood periods may range from 160 cfs to 320 cfs. Therefore, a capacity of 3,400 cfs at S-319 would provide adequate protection to the western basin for a 1-in-10 year storm and would meet the current drainage allocation in the basin.

For the backpumping analyses, The runoff rates that were allocated for each inflow point in the basin were used, with the exception of gravity inflow structures

	Sub-basin Number:															
Days	1	2	3	4	5	6	7	8	9	10	11	12	13	14	16	17
1	12.00	12.00	13.00	13.50	12.00	13.30	13.00	14.00	15.00	16.00	14.00	15.00	13.00	13.00	14.00	14.00
2	12.50	12.30	13.50	14.60	13.00	13.80	13.50	15.00	15.50	17.50	15.00	16.00	14.50	14.00	14.50	15.50
3	12.00	12.00	13.00	14.00	13.25	13.00	13.50	14.00	15.00	17.25	14.60	15.00	13.50	13.80	14.00	15.00
4	13.10	12.00	13.25	14.60	13.35	13.50	14.00	14.50	15.00	17.40	15.50	16.00	14.80	14.50	15.50	16.50
5	15.25	14.40	14.40	16.40	14.65	15.10	15.80	16.90	17.60	20.50	19.32	20.15	17.10	16.95	17.40	17.60
6	15.00	15.60	15.50	16.15	15.50	15.00	15.55	16.75	17.47	20.28	19.33	19.85	17.08	16.90	17.41	17.61
7	14.85	15.80	15.70	15.88	15.70	14.85	15.30	16.65	17.30	20.00	19.34	19.65	16.92	16.80	17.40	17.62
8	14.75	15.65	15.57	15.55	15.60	14.68	15.05	16.50	17.15	19.65	19.35	19.45	16.70	16.72	17.10	17.25
9	14.45	15.50	15.44	15.20	15.55	14.50	14.65	16.33	16.95	19.20	19.30	19.35	16.60	16.65	16.50	16.80
10	14.00	15.35	15.31	14.85	15.48	14.25	14.45	16.15	16.45	18.90	19.18	19.20	16.52	16.55	16.35	16.40
11	13.60	15.20	15.18	14.50	15.40	14.10	14.40	15.85	16.00	18.40	19.05	19.10	16.35	16.45	16.30	16.30
12	13.25	15.05	15.05		15.30	14.00		15.50		18.00	18.95	19.05	16.20	16.35	16.25	16.20
13	13.00	14.90	14.92		15.20			15.30		17.50	18.88	18.90	16.05	16.20	16.20	16.10
14	12.80	14.75	14.79		15.10			15.25		17.25	18.80	18.70	15.65	16.05		16.00
15		14.60	14.66		15.00			15.15			18.65	18.50	15.25	15.95		
16			14.53		14.90						18.55	18.30				
17					14.80						18.45					
18					14.70											
19					14.60											
20					14.50											
21					14.40											
22					14.30											
23																
24																
25																
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30																

Table A-9. Flood Elevations (ft NGVD) and Durations for Sub-basins of the Western C-51 Basin During a 1-in-10 Yr Storm Event. Royal Palm Beach Sub-basin 15 Excluded.

	Sub-basin Number:												
Days	1	2	3	4	5	6	7	8	9	10	11	12	13&14
1	12.00	11.50	13.00	14.00	13.00	13.00	13.00	14.00	15.00	17.00	15.00	17.50	12.50
2	12.50	12.00	13.40	14.00	13.30	13.50	13.50	14.50	15.70	17.50	16.00	18.20	13.50
3	13.00	12.10	13.30	14.50	13.40	13.50	13.40	15.00	15.85	18.30	16.50	18.25	14.50
4	13.50	12.30	13.55	15.00	13.85	14.20	14.50	15.20	16.35	19.05	17.50	18.90	15.65
5	16.65	14.90	15.10	18.00	15.25	15.75	16.55	17.50	18.70	21.00	20.05	21.00	17.50
6	18.00	16.00	15.90	18.30	15.85	15.90	16.40	17.40	18.70	20.50	20.05	20.65	17.45
7	18.05	16.60	16.50	18.25	16.40	16.40	16.25	17.33	18.50	20.25	20.05	20.40	17.40
8	18.10	16.90	16.80	18.18	16.75	16.65	16.05	17.26	18.25	20.05	20.06	20.22	17.35
9	17.90	17.00	16.90	18.10	16.85	16.72	15.90	17.19	18.10	19.65	20.07	19.95	17.30
10	17.75	16.86	16.78	18.10	16.77	16.64	15.70	17.12	17.95	19.10	20.08	19.77	17.20
11	17.61	16.72	16.65	17.72	16.69	16.56	15.45	17.05	17.88	18.95	20.10	19.65	17.12
12	17.40	16.58	16.53	17.65	16.61	16.48	15.00	16.98	17.73	18.00	20.10	19.55	17.05
13	17.30	16.44	16.40	17.55	16.53	16.40	14.60	16.91	17.60	17.00	20.06	19.50	17.00
14	17.15	16.30	16.28	17.40	16.45	16.32	14.30	16.84	17.45		20.05	19.44	16.95
15	16.95	16.16	16.15	17.25	16.37	16.24		16.77	17.30		20.00	19.35	16.90
16	16.80	16.02	16.03	17.10	16.29	16.16		16.70	17.15		19.95	19.29	16.85
17	16.65	15.88	15.90	17.05	16.21	16.08		16.63			19.90	19.24	16.80
18	16.50	15.74	15.78	16.95	16.13	16.00		16.56			19.85	19.20	
19	16.35	15.60	15.65	16.85	16.04	15.92		16.49			19.80		
20	16.20	15.46	15.53	16.70	15.96	15.84		16.42			19.75		
21	16.05	15.32	15.40	16.60	15.88	15.76		16.35			19.70		
22	15.80	15.18	15.28	16.47	15.80	15.68		16.27			19.65		
23	15.75	15.04	15.15	16.34	15.71	15.60		16.20			19.60		
24	15.60	14.90	15.03	16.21	15.62	15.52		16.13			19.55		
25				16.08	15.54	15.44							
26													
27													
28													
29													
30													

Table A-10. Flood Elevations (ft NGVD) and Durations for Sub-basins of the Western C-51 Basin During a 1-in-100 Yr Storm Event. Royal Palm Beach Sub-basin 15 Excluded.

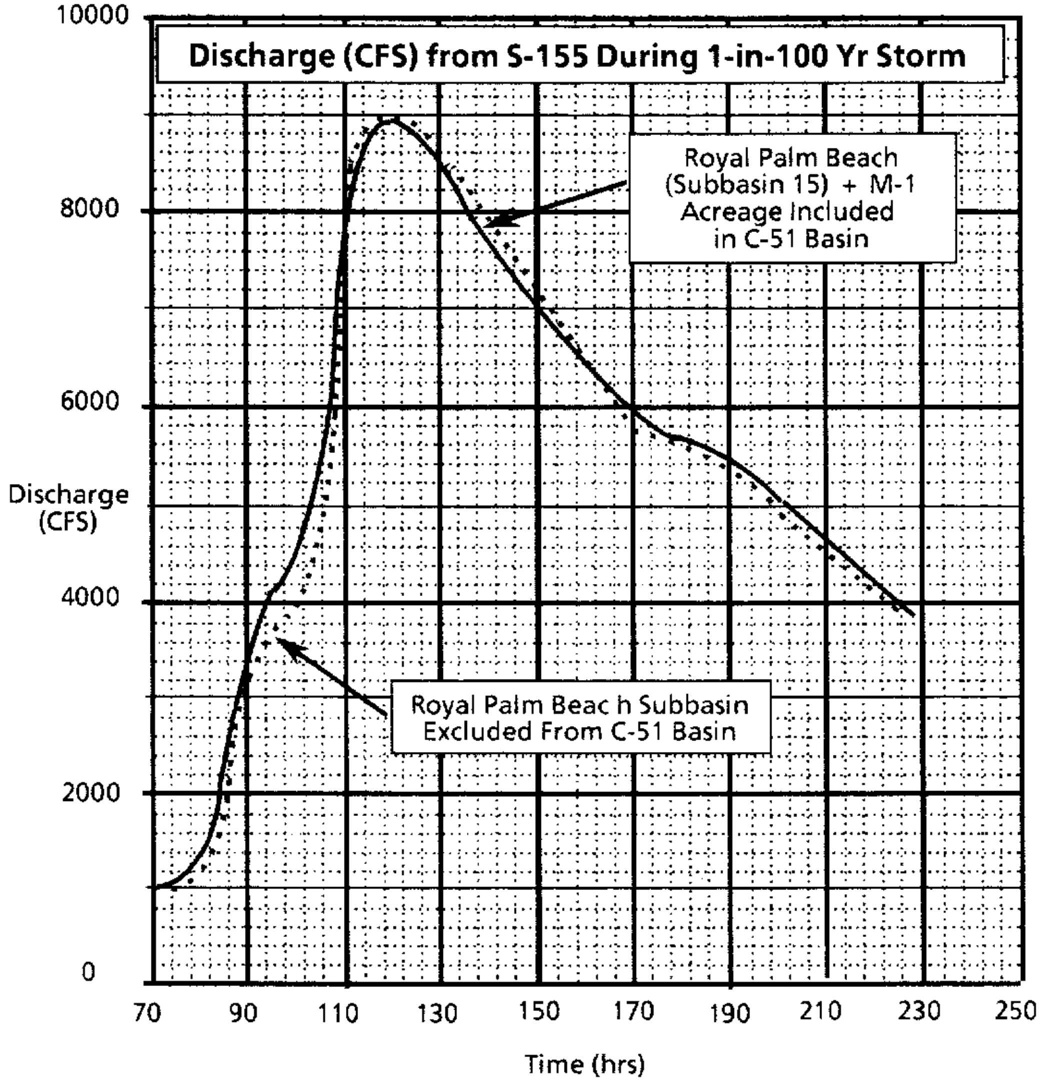


Figure A-12. Discharge Hydrograph from Structure S-155 during a 1-in-100 Yr Storm Event, with and without Royal Palm Beach and the M-1 Acreage Included in the Basin--Existing and Committed Land Use.

such as culverts, weirs, amil gates, etc. (see Table A-1). The inflows from these latter types of structures would be greater under the backpumping plan than without the plan because the available head across the structure would be increased. A slight increase in flow was assumed for these structures. For example, 720 cfs was used for the Amil gate at Royal Palm Beach during the 1-in-10 year storm, 780 cfs for the 1-in-25 year storm, and 1,000 cfs for the 1-in-100 year storm. This increase was due to the fact that there were two 72" x 95' CMP pipes that discharge directly into C-51. Even though these culverts are sealed by brick to 13.5 ft NGVD, the maximum discharge capacity from the Amil gate under 3 ft or greater head would be 1,500 cfs (based on the rating of the Amil gate). Therefore, two scenarios were investigated--one based on restricted (allocated) flow of up to 1,000 cfs, and a second scenario based on the rating of the Amil gate structures (scenarios 4 and 5).

Scenarios that Include Backpumping.

A. Scenario 4. Backpumping Plan with the M-1 Basin Project completed and flow restrictions placed on the Amil gates at Royal Palm Beach.

The results of this scenario indicated that flow would not occur over the south bank of C-51 into the western sub-basins. Estimated flood stages in sub-basins 2 and 6 for this scenario were included in Figures A-10 and A-11, respectively, and in Figure A-9 for sub-basin 15 (Royal Palm Beach). Flood stages and durations in the western sub-basins are shown in Tables A-11 through A-13. The flood stage in Royal Palm Beach peaked at 18.25 and 18.90 ft NGVD during the 1-in-10 and 1-in-100 year storms. These stages were 0.3 to 0.35 ft less than peak stages in scenario 2. Duration of flooding was much shorter under the backpumping plan. Figure A-13 shows the water levels in C-51 for the 1-in-10 and 1-in-100 year storms without consideration of seepage flow. The backwater profile for the 1-in-100 year storm is much higher than the other storm event because the total runoff of 3,682 cfs from the western basin exceeds the pump capacity of 3,400 cfs. This excess inflow would be temporarily stored in the canal and would increase the canal stage until local inflows were less than 3,400 cfs.

B. Scenario 5. Backpumping Plan with the M-1 Project Completed and No Flow Restrictions on the Amil Gates at Royal Palm Beach.

Gravity inflows may be substantially increased due to the lower stage in C-51 under the backpumping plan. Outflow from the Amil gate structures at Royal Palm Beach could therefore be larger than the permitted allocation for this sub-basin. Scenario 5 investigated the impact of these higher flows through the Royal Palm Beach Amil gate structures, with the outflow from the remaining structures in the western basin restricted to their allocated flow levels.

Analysis of outflow hydrographs from Royal Palm Beach with the acreage area excluded (Figure A-14) indicated that peak discharges were 930 cfs and 1,500 cfs for the 1-in-10 and 1-in-100 year storms, respectively. Total runoff from the western C-51 basin was 3,935 cfs for the 1-in-100 year storm. Since 3,935 cfs exceeds the pump capacity, S-319 would have to be operated at its maximum capacity of 3,400 cfs from hours 109 through 175 (see Figure A-15). The backwater stage at Royal Palm Beach could reach 17.8 ft NGVD. The peak flood stage in Royal Palm Beach would drop 0.05 to 0.10 ft as compared with scenario 4; however, the flood waters would dissipate more rapidly.

C. Scenario 6. Backpumping Plan with M-1 Acreage Included in the C-51 Basin

The M-1 acreage was included in the C-51 basin and no outflow restrictions were applied to the Royal Palm Beach outfall in this scenario. The results indicated that no impact would occur to other basins. Total outflow from the western C-51 basin would be 4,000 cfs, which would cause higher stages in C-51 under a 1-in-100 year storm event. The major differences in flood stages in Royal Palm Beach under scenarios 4, 5, and 6 are presented in Table A-14.

Flood stages in Royal Palm Beach would be substantially reduced if the M-1 acreage were excluded from the basin (scenarios 4 and 5). By comparison, unrestricted operation of the Amil gate structure at Royal Palm Beach has minimal impact on flood stages in this sub-basin.

	Sub-basin Number:																			
Days	1	2	3	4	5	6	7	8	9	10	11	12	13&14	15	16	17	18	19	20	22
1	12.00	12.00	13.60	13.50	13.00	13.50	14.00	14.00	15.00	16.00	14.00	15.00	13.00	13.05	15.00	14.00	13.00	14.90	14.00	13.00
2	12.50	12.50	13.50	14.00	13.00	14.00	14.50	15.00	15.50	17.50	15.00	16.00	13.50	13.15	15.20	15.50	13.80	15.30	14.20	13.80
3	12.70	12.00	12.50	13.50	13.25	13.00	14.00	14.00	15.00	17.25	14.00	15.00	13.73	13.10	15.00	15.00	14.50	15.70	14.10	13.30
4	13.00	12.40	13.50	14.60	13.35	13.50	14.00	14.50	15.00	17.40	15.40	16.00	14.55	13.40	15.25	16.50	16.20	16.10	14.90	14.00
5	14.90	13.45	14.50	16.80	14.60	15.15	15.80	16.90	17.60	20.50	19.15	20.15	17.00	18.25	17.25	17.60	17.35	17.10	17.00	18.30
6	14.70	13.25	14.15	16.60	14.50	14.95	15.55	16.75	17.40	20.25	19.00	19.85	16.95	16.65	16.90	17.61	16.72	16.35	16.30	17.80
7	14.40	13.00	14.00	16.30	14.40	14.80	15.30	16.65	17.20	19.95	18.75	19.60	16.90	14.50	16.30	17.25	16.25	16.00	15.65	17.10
8	14.15	12.60	13.75	16.05	14.25	14.65	15.05	16.50	16.95	19.50	18.50	19.40	16.80	13.55	16.20	16.80	16.10	15.90	15.05	15.20
9	13.75	12.30	13.50	15.90	14.10	14.50	14.65	16.33	16.30	19.10	18.25	19.25	16.70	13.25	16.10	16.40	16.00		14.50	14.40
10	13.50	12.10		15.75	13.95	14.20	14.45	16.15	15.90	18.75	17.90	19.10	16.60	13.10		16.30				
11	13.25			15.50	13.75	14.00	14.40	15.85		18.00	17.60	18.95	16.50			16.20				
12	13.00			15.25	13.55		14.35.	15.50		17.80	17.50	18.75	16.40			16.10				
13				15.00	13.45			15.35		17.50	17.40	18.45	16.30							
14					13.30			15.30			17.30	18.20	16.20							
15												18.00								

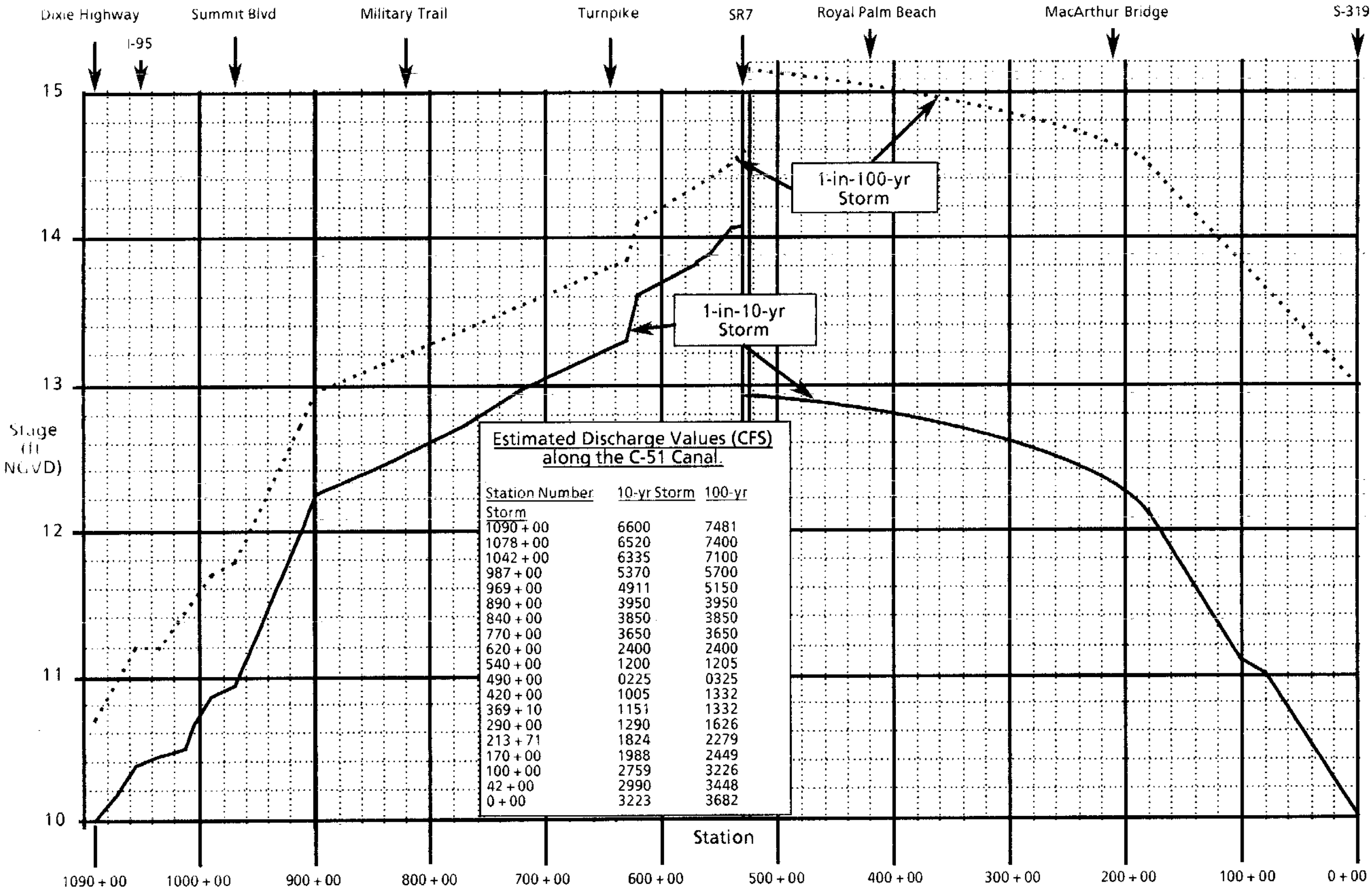
Table A-11. Flood Elevations (ft NGVD) and Durations for Sub-basins of the Western C-51 Basin During a 1-in-10 Yr Storm Event. Backpumping Plan with M-1 Project Completed

	Sub-basin Number:																			
Days	1	2	3	4	5	6	7	8	9	10	11	12	13&14	15	16	17	18	19	20	22
1	12.00	12.00	13.00	14.00	13.00	13.20	14.00	14.00	15.00	17.50	14.00	15.00	13.00	13.20	15.00	14.50	13.10	15.00	14.10	13.10
2	12.50	12.10	13.30	14.50	13.30	13.50	14.10	14.50	15.50	18.10	15.00	16.20	14.00	13.40	15.50	15.50	13.90	15.50	14.25	14.00
3	12.80	12.00	13.00	14.25	13.40	13.00	13.50	14.00	15.80	17.60	15.00	15.10	14.20	13.50	15.40	15.40	14.60	16.00	14.00	14.30
4	13.10	12.20	13.40	14.70	13.60	13.60	14.20	14.70	16.00	18.30	16.50	18.35	15.10	14.00	16.00	16.20	16.30	16.20	15.10	15.50
5	15.95	13.65	14.80	16.75	14.90	15.40	16.05	17.25	18.00	20.70	17.50	20.45	17.40	18.60	17.50	17.80	17.60	17.45	17.35	18.40
6	15.85	13.50	14.50	16.55	14.80	15.30	16.00	17.15	17.85	20.48	17.45	20.20	17.30	17.85	17.25	17.60	17.10	16.95	16.55	18.30
7	15.70	13.30	14.35	16.35	14.65	15.15	15.75	17.05	17.65	20.22	17.40	19.90	17.25	15.50	16.85	17.40	16.45	16.35	15.90	17.85
8	15.50	13.10	14.15	16.05	14.55	15.05	15.50	16.95	17.45	19.90	17.35	19.65	17.20	15.00	16.40	17.10	16.20	16.10	15.00	17.10
9	15.30	12.80	13.90	15.75	14.45	14.90	15.25	16.85	17.28	19.45	17.30	19.50	17.10	14.50	16.30	16.85	16.05	16.00	14.50	15.40
10	15.10	12.45	13.70	15.45	14.30	14.70	14.90	16.70	17.05	18.90	17.25	19.35	17.00	13.55	16.20	16.45	15.90	15.90		
11	14.85	12.25	13.50	15.00	14.10	14.50	14.60	16.55	16.65	18.40	17.20	19.30	16.90		16.10	16.25				
12	14.70	12.20		14.80	13.95	14.35	14.30	16.35	16.10	18.00	17.15	19.20	16.80		16.00	15.95				
13	14.50	12.00		14.60	13.75	14.15	14.00	16.15	15.50	17.52	17.10	19.10	16.70			15.60				
14	14.30				13.50	14.00		15.95			17.05	19.00	16.60							
15																				

Table A-12. Flood Elevations (ft NGVD) and Durations for Sub-basins of the Western C-51Basin During a 1-in-25 Yr Storm Event. Backpumping Plan with M-1 Project Completed.

	Sub-basin Number:																		
Days	1	2	3	4	5	6	7	8	9	10	11	12	13&14	15	16	17	18	19	20	22
1	12.70	12.00	13.00	14.00	13.00	13.00	13.00	14.00	15.00	17.80	15.00	15.00	12.50	13.10	14.00	14.50	13.50	15.00	14.10	13.20
2	12.80	12.10	13.40	14.00	13.30	13.50	13.50	14.50	15.70	18.20	16.00	16.10	13.00	13.25	14.50	15.00	14.00	15.50	14.30	14.20
3	13.10	12.20	13.30	14.50	13.40	13.40	13.40	15.00	15,85	18.40	16.50	15.00	14.40	13.35	15.00	16.20	15.50	15.90	14.50	14.90
4	13.40	12.35	13.50	14.80	13.80	13.80	14.10	15.20	16.40	19.00	17.30	18.80	15.65	14.90	16.00	16.70	16.50	16.40	15.85	15.90
5	15.90	14.05	15.10	18.00	15.30	15.75	16.55	17.50	18.70	21.00	20.05	20.75	17.50	18.90	17.85	18.05	17.90	17.80	17.80	18.90
6	16.70	13.85	14.95	17.60	15.20	15.70	16.40	17.40	18.70	20.50	19.95	20.60	17.45	18.25	17.60	18.10	17.55	17.40	17.15	18.75
7	17.00	13.70	14.80	17.20	15.15	15.65	16.25	17.30	18.10	20.20	19.85	20.30	17.35	16.40	17.30	17.75	17.00	16.75	16.45	18.50
8	16.90	13.55	14.65	16.90	15.05	15.55	16.05	17.20	17.90	19.95	19.75	20.10	17.25	14.40	17.00	17.35	16.40	16.25	15.80	18.20
9	16.80	13.40	14.50	16.70	14.95	15.40	15.90	17.10	17.75	19.55	19.65	19.95	17.15	13.60	16.60	17.20	16.20	16.05	15.10	17.60
10	16.70	13.25	14.35	16.50	14.85	15.35	15.70	17.00	17.50	19.00	19.55	19.80	17.05		16.50	17.00	16.10	15.90	14.50	16.60
11	16.55	13.10	14.20	16.30	14.75	15.25	15.45	16.90	17.30	18.50	19.45	19.70	16.95		16.40	16.70	16.00			15.75
12	16.40	12.50	14.00	16.10	14.65	15.10	15.00	16.80	17.00	18.00	19.35	19.60	16.85		16.30	16.50				15.50
13	16.30	12.30	13.80	15.90	14.55	14.95	14.60	16.70	16.50	17.50	19.25	19.50	16.75		16.20	16.30				15.25
14	16.15	12.20	13.70	15.70	14.45	14.80	14.30	16.60	16.00	17.00	19.15	19.40	16.65		16.10	16.15				15.00
15	16.05	12.10	13.60	15.50	14.35	14.65		16.50	15.70	16.70	19.05	19.30	16.55		16.00	16.00				

Table A-13. Flood Elevations (ft NGVD) and Durations for Sub-basins of the Western C-51 Basin During a 1-in-100 Yr Storm Event. Backpumping Plan with M-1 Project Completed



Estimated Discharge Values (CFS) along the C-51 Canal.

Station Number	10-yr Storm	100-yr Storm
1090 + 00	6600	7481
1078 + 00	6520	7400
1042 + 00	6335	7100
987 + 00	5370	5700
969 + 00	4911	5150
890 + 00	3950	3950
840 + 00	3850	3850
770 + 00	3650	3650
620 + 00	2400	2400
540 + 00	1200	1205
490 + 00	0225	0325
420 + 00	1005	1332
369 + 10	1151	1332
290 + 00	1290	1626
213 + 71	1824	2279
170 + 00	1988	2449
100 + 00	2759	3226
42 + 00	2990	3448
0 + 00	3223	3682

Figure A-13 . Expected Water Elevations (Ft NGVD) and Discharge Rates (CFS) in the C-51 Basin During 1-in-10 and 1-in-100 yr Storm Events. Future Land Use Conditions with Backpumping of the Western Basin.

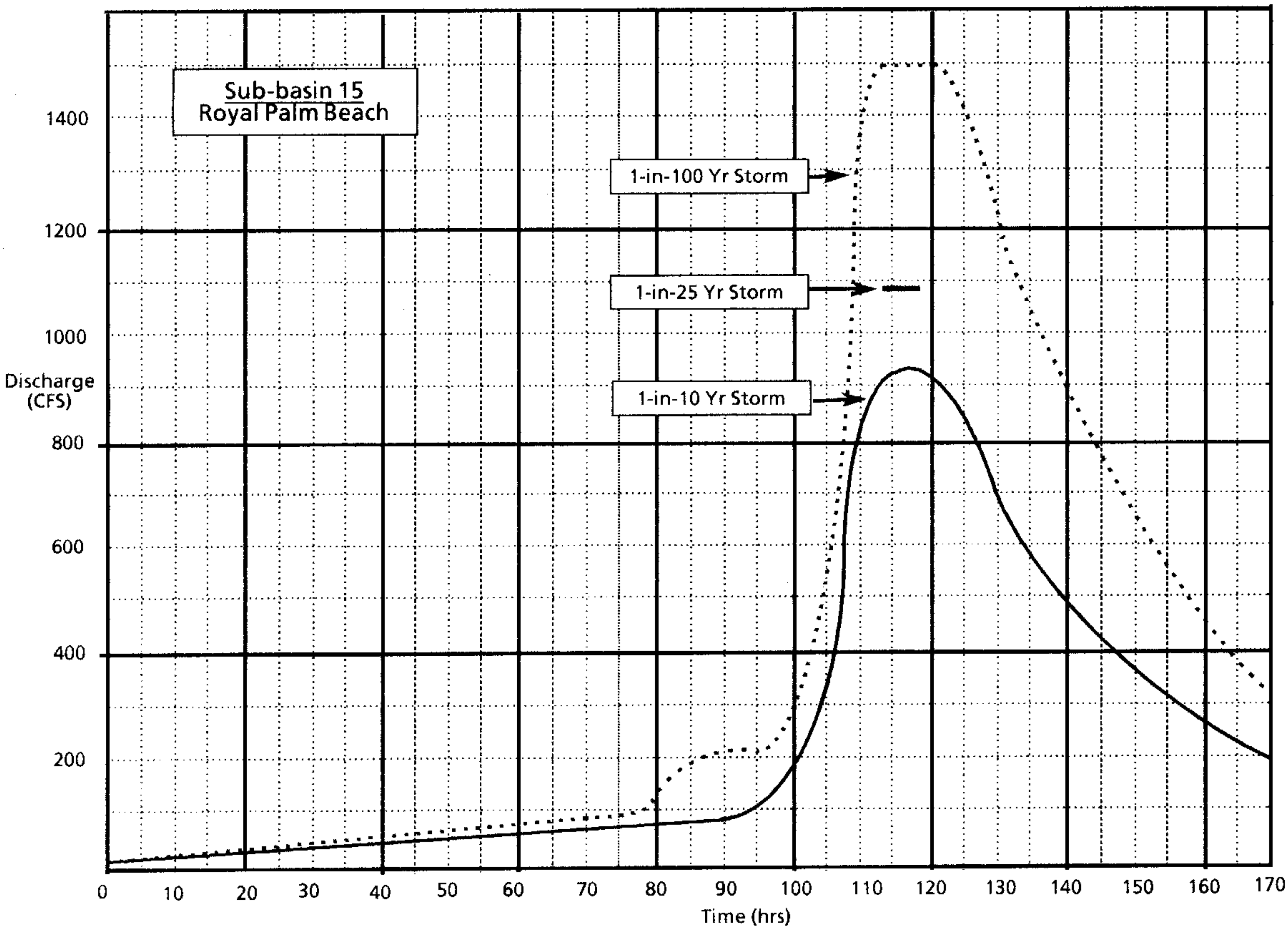


Figure A-14. Outflow Hydrographs for 1-in-10 and 1-in-100 Yr Storm Events and Peak Discharge for the 1-in-25 Yr Event at Outfall of Royal Palm Beach (Sub-basin 15) during Backpumping, with No Outfall Restrictions.

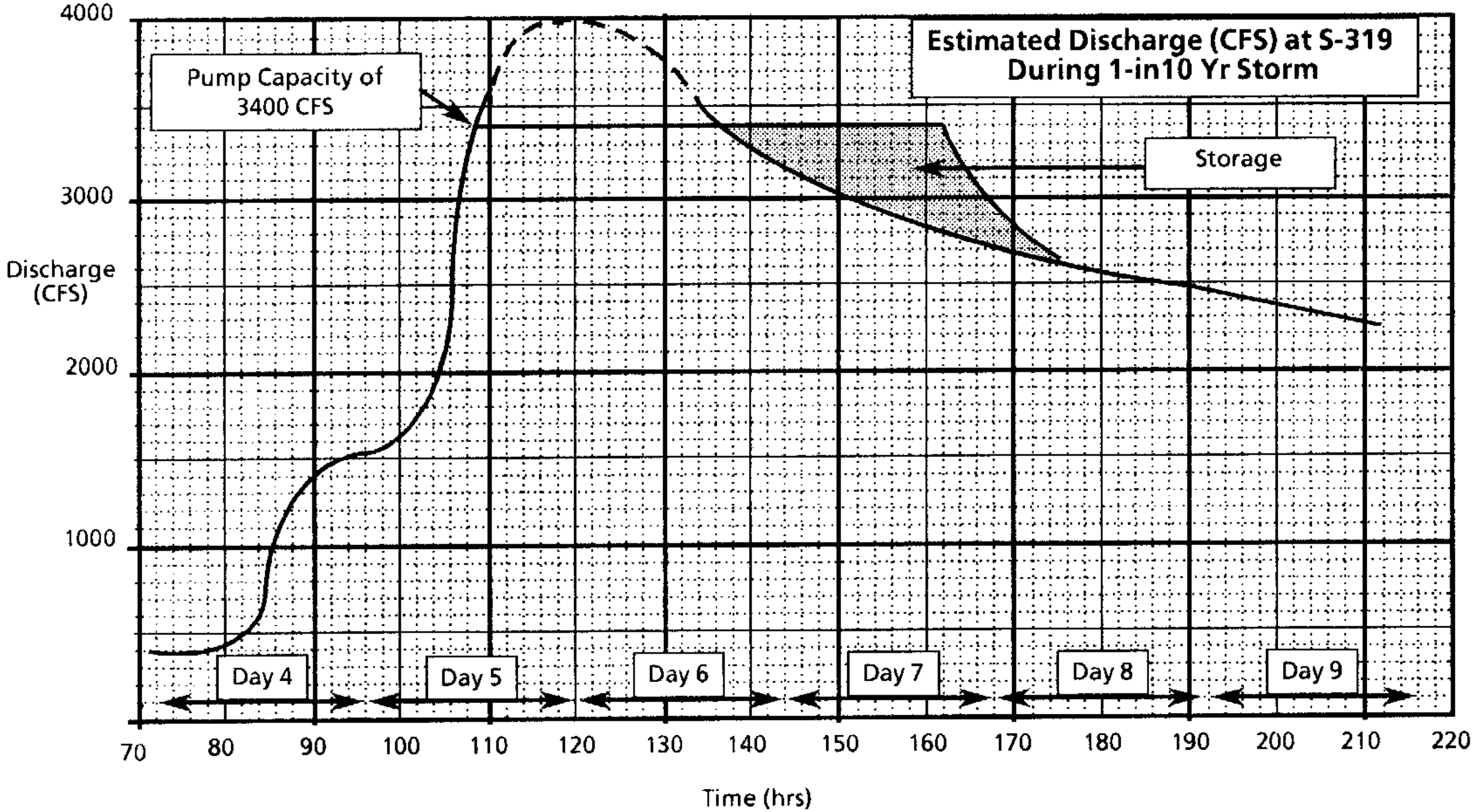


Figure A-15. Discharge Hydrograph at S-319 during a 1-in-10-Yr Storm, with no Restrictions on the Amil Gate at Royal Palm Beach and Backpumping Capacity of 3400 CFS.

TABLE A-14. Stages (Ft NGVD) in Royal Palm Beach with Backpumping Plans

	10-Year Storm			100-Year Storm		
Scenario No:	(4)	(5)	(6)	(4)	(5)	(6)
Day						
1	13.05	13.05	13.10	13.10	13.10	13.15
2	13.15	13.15	13.35	13.25	13.25	13.40
3	13.10	13.10	13.20	13.35	13.30	13.80
4	13.40	13.35	14.00	14.90	14.90	17.20
5	18.25	18.20	19.15	18.90	18.80	19.90
6	16.65	16.30	18.30	18.25	17.85	19.40
7	14.50	14.30	16.90	16.40	16.40	18.70
8	13.55	13.45	15.00	14.40	14.10	17.75
9	13.25	13.25	14.05	13.60	13.40	15.60
10	13.10	13.05	13.50	13.30	13.25	14.20

Impacts of Backpumping on Flood Conditions in the Eastern Basin

The assumptions that were used in the analysis of the eastern basin included the following:

a. Backpumping
b. Future land use
c. Channel improvements as proposed by the USCOE in the Detailed Design Memorandum for C-51.
d. Allowable runoff allocations for the Lake Worth Drainage District, based on existing capacities of outlet structures in the basin.

The results of this case were presented in Figure A-13, which shows the peak water levels (ft. NGVD) and discharge rate (cfs) in eastern C-51 under 1-in-10 and 1-in-100 year storm events. This figure should be compared with Figure A-8, which shows water stages in the basin without backpumping. With the western portion of the C-51 basin backpumped, the peak discharges at S-155 were 6,600, 7,155, and 7,481 cfs during the 1-in-10, 1-in-25, and 1-in-100 year storm events respectively. Figure A-16 shows the hourly rainfall distribution for the 1-in-100 year storm and the discharge hydrograph at S-155. Runoff at S-155 reached its peak in 10 to 12 hours after the most intensive rainfall (5.78 in/hr before noon of the 5th day) in the 1-in-100 year storm event. The discharge hydrograph under the backpumping scheme receded more rapidly than the hydrograph for the present condition without backpumping (Figure A-12). The flood stage that occurs in C-51 with backpumping was also considerably lower than the stage that occurs without backpumping. For example, the flood stages at Summit Boulevard dropped to 10.94 and 11.80 ft NGVD under backpumping for the 1-in-10 and 1-in-100 year storms respectively (see Figures A-8 and A-15 for comparisons with and without backpumping). Likewise, peak stages at SR7 would be 14.08 and 14.58 ft NGVD under backpumping as compared to 17.96, and 18.26 ft NGVD under the present conditions without backpumping.

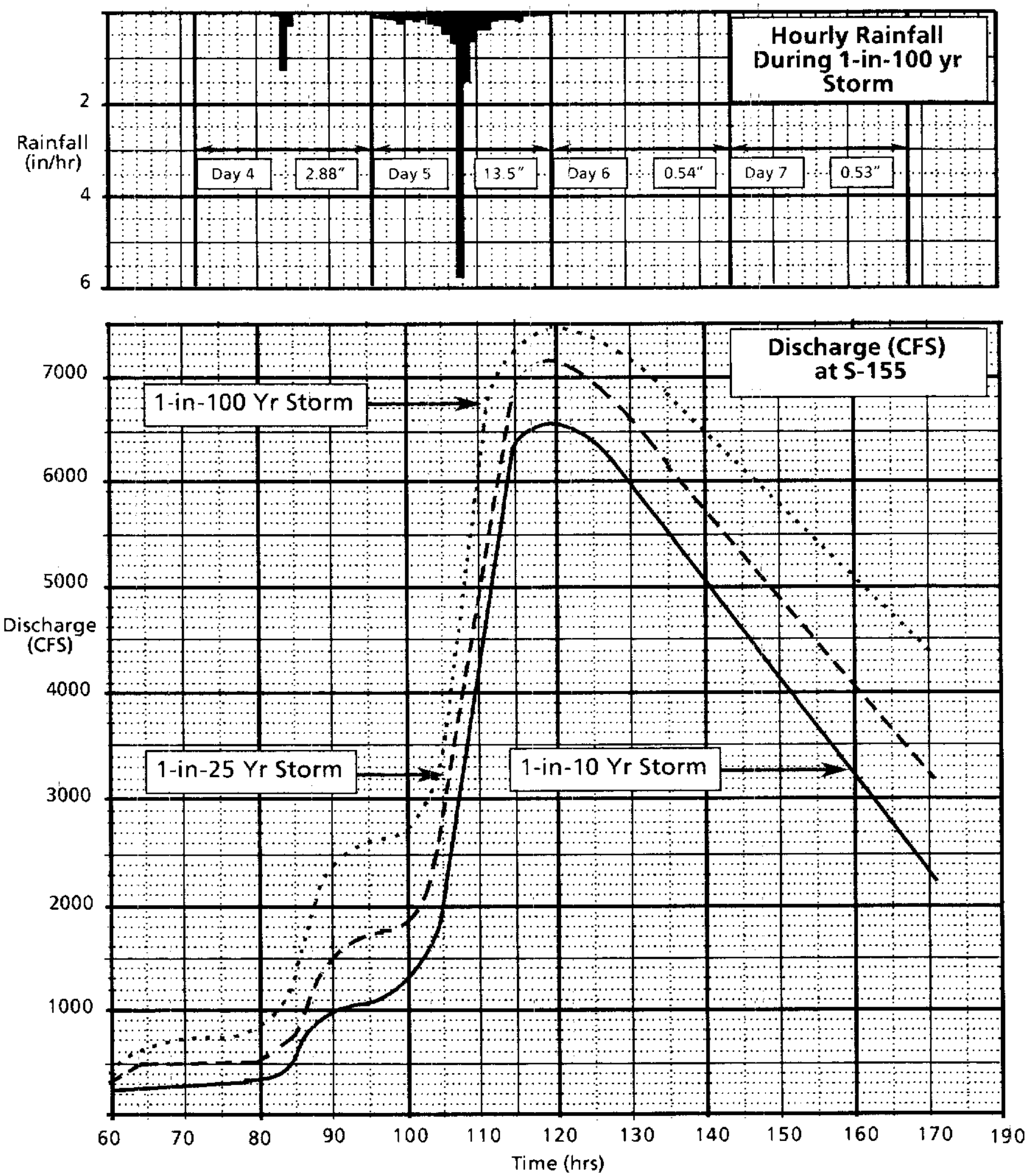


Figure A-16. Rainfall (in/hr) and Discharge Hydrographs for the 1-in-10, 1-in-25 and 1-in-100 Yr Storm Events at S-155, with Backpumping and the Improved Channel East of SR7.

Appendix A-Supplement

Equations

$$K=\frac{dlogQ}{dt} \qquad (1)$$

By integration,

$$logQ=Kt+C$$

Solve for C

$$\because\ t=t_n \qquad Q=Q_n$$

at W_{50} in cfs

$$\therefore\ logQ_n=Kt_n+C$$

$$C=logQ_n-Kt_n$$

Therefore:

$$logQ=Kt+logQ_n-Kt_n=logQ_n+K(t-t_n)$$

$$logQ-logQ_n=K(t-t_n)$$

or

$$\frac{Q}{Q_n}=e^{K(t-t_n)}$$

$$\therefore\ Q=Q_ne^{K(T-t_n)} \qquad (2)$$

Let RV = remaining volume under hydrograph from time n expressed in cfs-hrs

$$RV=\int_{t_n}^{\infty}Qdt=\int_{t_n}^{\infty}Q_ne^{K(t-t_n)}dt=Q_n\int_{t_n}^{\infty}e^{K(t-t_n)}dt$$

$$=Q_ne^{-Kt_n}\int_{t_n}^{\infty}e^{Kt}dt=Q_ne^{-Kt_o}\frac{1}{K}e^{Kt}\Big|_{t_n}^{\infty}$$

$$=Q_ne^{-Kt_n}\frac{1}{K}(e^{K\infty}-e^{Kt_n})$$

If Q is decreasing, then K is a negative value

$$RV=Q_ne^{-Kt_n}\cdot\frac{1}{K}(e^{Kt_n})=\frac{-Q_n}{K}$$

$$\therefore\ K=-\frac{-Q_n}{RV} \qquad (3)$$

$$\therefore\ Qt_{n+1}=Q_ne^{-\frac{Q_n}{RV}(t-t_n)} \qquad (4)$$

WATER MANAGEMENT PLANNING FOR THE WESTERN C-51 BASIN

March 1984

Appendix B
WATER QUALITY EVAULATION OF THE C-51 BASIN

Prepared by
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South Florida Water Management District
West Palm Beach, Florida

Appendix B

WATER QUALITY EVALUATION OF THE C-51 BASIN

Introduction

The purpose of the water quality evaluation of the C-51 basin is to address four major areas as follows: 1) Land use and other environmental factors that may affect water quality in C-51; 2) Estimated water quality at the proposed S-319 pump station; 3) Comparison of water quality at the proposed pump station with existing WCA-1 inflows and water quality inside WCA-1; and 4) Estimated effects of S-319 pumpage on nutrient loadings into WCA-1.

Data Sources. The water chemistry data used in this report were extracted from a variety of sources whose study objectives were not related to this evaluation. These raw data represent the best available data in lieu of a time-consuming and costly, site-specific study. Because these data were collected by a variety of public and private entities, the data sets are not parallel as to period of record or method. Table B-1. shows the various sources of data, sampling periods, number of observations per site, and the total number of observations per parameter group.

TABLE B-1. WATER CHEMISTRY DATA SOURCES FOR DATA USED IN TABLES IN THE C-51 BACKPUMPING EVALUATION

Data Set	Agency	Date Range	# of Sample Sites	Physical	Nutrients	Major Ions	Heavy Metals	Pesticide/ Herbicides	Coliform
WCA1 inflow	SFWMD	1/1/78-12/31/80	5	78	78	78	6	0	0
WCA1 interior	SFWMD	5/1/78-7/31/83	21	14	14	14	3	0	0
S5AE	SFWMD	3/24/82-11/9/82	1	9	10	3	0	0	0
C-51 at SR7	SFWMD	6/28/74-1/17/77	1	19	20	20	6[3]	0	0
Indian Trail	Joyce E.C.	2/16/83-6/21/83	3	5	5	5	5[3]	2	5
Callery Judge Grove	C.J. Grove	1/5/79-4/25/83	3	0	26[4]	0	26[3]	26	26
Sugar Cane	SFWMD	1/1/78-6/30/83	2	74	92	3	3	0	0
Citrus	SFWMD	11/1/73-10/31/74	6	0	26	26	0	0	0

[1]During flow to the west only
[2]During flow to the east and S-5AE closed
[3]Copper analysis only
[4]TPO_4 and NO_3 analysis only

SFWMD Data. All collections by the SFWMD were performed using a Nalgene collector from 0.0 to 0.5 meters depth. Samples were refrigerated prior to analysis. All analyses were done according to standard methods and the laboratory is certified by the Department of Environmental Regulation (DER). Water chemistry data from other sources were similarily collected and analyzed.

Factors Affecting Water Quality in C-51.

Land Use. The quality of water in C-51 is the result of the quality of runoff from the varied land uses in the basin. Table B-2 gives a general summary of the land uses

TABLE B-2. MAJOR PERMITTED LANDHOLDINGS IN THE C-51 BASIN

	Land Holding	Land Use	Est. Area (mi 2)
NORTH of C-51	Indian Trails (M-1)	Rural/Res.	6.0
	Village of Royal Palm Bch.	Residential	6.5
	Loxahatchee WCD	Rural/Res.	14.0
	Seminole WCD	Citrus	6.25
	Indian Trails (M-2)	Rural/Res.	7.10
	Lion Country Safari	Recreational	1.03
	Double D. Ranch	Pasture	1.92
	L-8 Ranch	Pasture	1.48
	Fox Trail	Rural/Res	1.75
	Entrada Acres	Vegetables	0.30
	Guest Farms	Vegetables	0.40
	Misc.	Mixed	3.5
SOUTH of C-51	Juicy Orange Groves	Citrus	1.25
	Busbee and Wilkins	Citrus	1.25
	Sucrose Growers	Sugarcane	2.8
	Macklen Enterprises	Citrus	1.0
	McArthur Farms	Pasture/veg.	2.1
	Diamond C Sugar	Sugarcane	1.2
	Wellington	Mixed	15.0
	Totals		74.83

in the basin based on permits that have been issued by the SFWMD. Suburban residential developments (44.35 sq mi) comprise the majority of the land use in the C-51 basin west of SR-7. The order of abundance of the remaining land use types are as follows: citrus (9.75 sq mi) > pasture (4.45 sq mi) > sugarcane (4.0 sq mi) > vegetables (1.75 sq mi). Some estimates of the water quality for these land uses are presented in Table B-3. The water quality data in Table B-3 were used to develop a generalized ranking of the land use types based on runoff water quality parameters. This ranking may indicate future water quality trends as land use patterns change in the basin. In general the major land use types can be ranked from highest to lowest concentrations of nitrogen and phosphorus as follows:

Total nitrogen--sugar cane > pasture > citrus > suburban

Total phosphorus--pasture > suburban > sugarcane > citrus.

Sludge Disposal. Another factor that may influence water quality in C-51 is the practice of sludge disposal for soil enrichment. Stabilized sludge from local municipal sewage treatment plants is spread on pastures and citrus groves under permit from the Department of Environmental Regulation and Palm Beach County Health Department (Figure B-1). Table B-4 shows the size and application rate of sludge disposal sites in the C-51 basin. The Palm Beach County Health Department has established three criteria for sludge disposal:

1. The sludge must be stabilized and have undergone secondary treatment to reduce pathogenic content and odor.
2. Coliform counts must have been reduced by a factor of one thousand
3. Sludge disposal is prohibited if the water table is less than 12 inches from the surface.

No runoff water quality data are available from these sludge disposal sites, for periods when the prescribed rules of application are being followed. However, data were collected during a three-day period in June 1982, at one disposal site, during a

TABLE B-3. COMPARISON OF WATER QUALITY OF RUNOFF FROM VARIOUS LAND USES WITHIN THE WESTERN C-51 DRAINAGE BASIN

Parameter	Suburban [1]	Sugarcane [2]	Citrus	Improved Pasture [4]
Conductance (µhos/cm)	497	849	-	133
Turbidity (NTU)	3.1	4.6	-	2.8
Color (units)	-	95	-	185
TPO_4 (mg/L)	0.08	.070	0.068[3A]	0.23
NO_3 (mg/L)	0.44	1.258	0.31[3A]	0.05
NH_4 (mg/L)	0.13	0.27	-	0.55
Organic N (mg/L)	0.63	2.87	1.06[3B]	1.4
Total N (mg/L)	0.63	2.87	1.06[3B]	2.0
Chloride (mg/L)	45	105	113[3B]	-
Copper (mg/L)	5.0	9.5	31.0[3A]	-

[1]Indian Trail - Joyce Environmental Consultants
[2]Lake Okeechobee Water Quality - SFWMD Technical Memo (January, 1983)
[3A]Callery Judge Grove - Applied Agricultural Research
[3B]Strazzulla Grove - IFAS Conference on Nonpoint Pollution Control Technology in Florida. March, 1982.
[4]Okeechobee County - SFWMD, unpublished data, A. Goldstein Upland Detention/ Demonstration Project - Final Report (in preparation)

heavy rainfall event. These data suggest that potentially high levels of nutrient enrichment of C-51 (especially for phosphorus) could occur if the proper application guidelines were not strictly followed. Therefore sludge disposal in the C-51 basin requires careful oversight.

Pesticides. The use of pesticides in the C-51 basin could be a potential source of these materials to WCA-1 due to S-319 backpumping. Several pesticides have been detected in runoff water in one citrus grove of the C-51 basin (Table B-5). Among those compounds that were frequently detected were kelthane (dicofil) - a miticide used on fruits, vegetables and ornamentals; chlorobenzilate - which is also used on citrus for the control of mites; and diuron - a herbicide that is used to control germinating weeds in sugarcane.

The vast majority of samples that indicated detectable levels of these chemicals were collected in the mid-1970's. In 1980, only 3 samples had detectable levels of kelthane, while ethion and lindane were each detected in one sample. Since 1980, monthly samples at 3 stations in this grove have shown no detectable levels of any pesticides or herbicides.

Samples of sediments that were collected at the same place and time as the water samples were also analyzed for pesticide and herbicide compounds. With the exception of DDT and DDE, the occurrence of detectable levels of pesticides or herbicides in sediments was less than the occurrence of these compounds in water. Some recent sediment samples (1981 and 1982) did, however, contain detectable residues of 2,4-D, kelthane and chlorobenzilate.

The results of a very limited sampling program for pesticides in one of the urban watersheds (Indian Trail) demonstrated no detectable levels for any of the compounds tested.

Estimate of S-319 Water Quality

The proposed S-319 pump station will serve the drainage area of C-51 from WCA-1 east to approximately S.R.7. Data collected for C-51 at S.R.7, during periods when flow was eastward and S-5AE was closed, represents the quality of

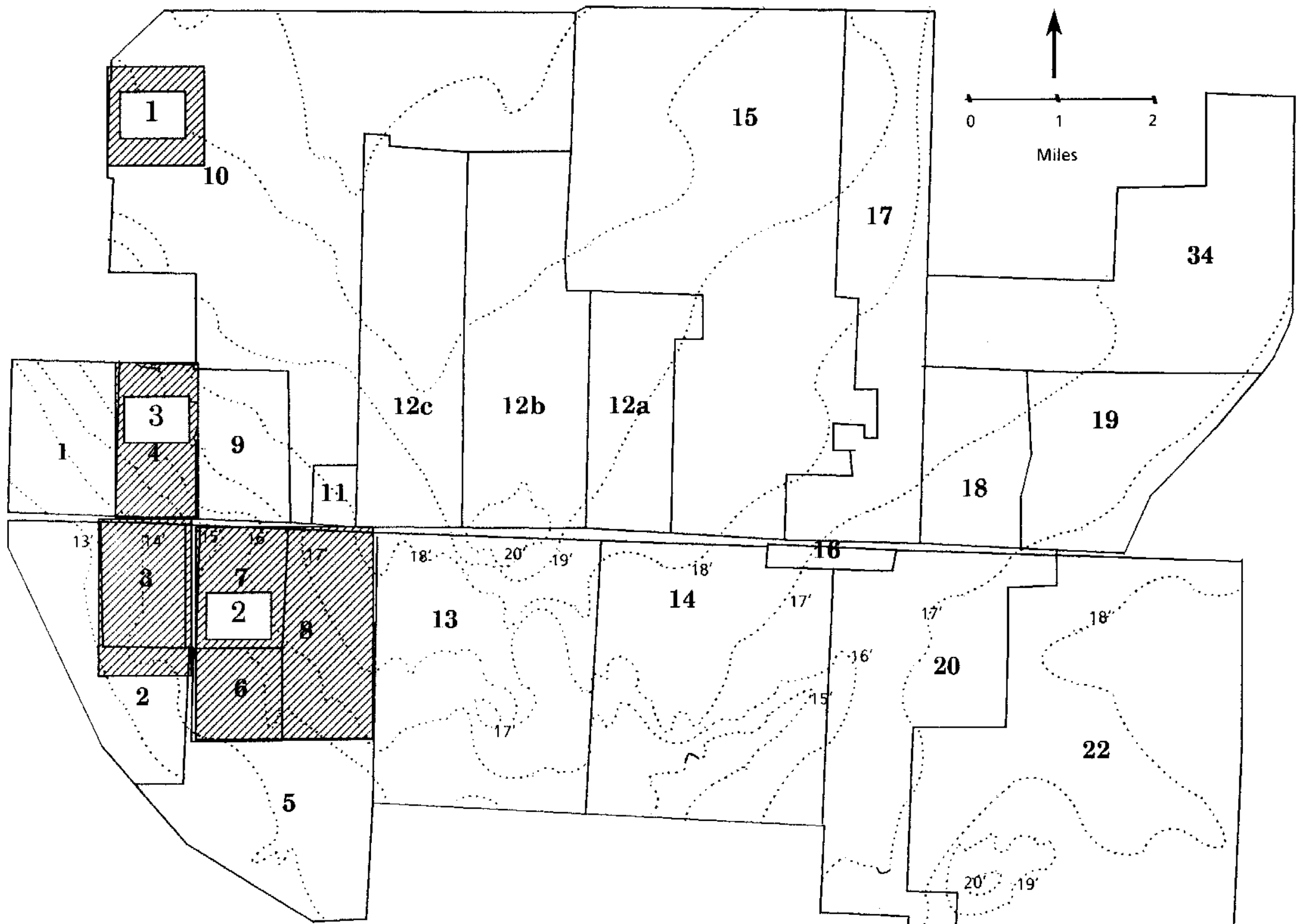

Figure B-1 Sludge Disposal Sites in the Western C-51 Basin

TABLE B-4. SLUDGE DISPOSAL SITES IN THE WESTERN C-51 DRAINAGE BASIN [1]

Map No.	Name	Acres	Application Rate (millions gals/yr)	Application Rate per Acre (millions gals/yr/acre)
1	Callery JudgeGroves	10	0.24	0.024
2	Juicy JuiceGrove	288	17.25	0.060
3	Sunshine Steers	560	68.0	0.121

[1] Data from Palm Beach County Health Department - July 1973.

TABLE B-5. OCCURRENCE OF PESTICIDE/HERBICIDES, CALLERY JUDGE GROVE SURFACE WATER AND SEDIMENTS 1972-82

	Year										
Compound:	1972	1973	1974	1975	1976	1977	1978	1979	1980	1981	1982
2,4-D	3W	2W		3W			2W			2S	
2,4,5-T (Silvex)		1W			2W					1W	
Diuron			1W	1W,6S	3W	9W	12W	7W,3S			
Dacthal							1W				
DDE				3S	5S	6S	1W,6S	6S	7S	5S	1S
DDT			1W			1S		3S			
Dieldrin		7W	9W	2S							
Aldrin			3W								
Diazinon		4W									
Parathion			2W								
Ethyl Parathion		6W		9W,6S	2W	2W					
Methyl Parathion		7W									
Heptachlor Epoxide			4W	1S							
Ethion						3W,2S	1S	3W	1W		
Phosdrin				2W,4S							
Phosphamidon				1W,1S							
Ronnel		3W									
Malathion		1W									
Kelthane			11W	16W,4S	14W,3S	2W,1S	2W,2S	8W,3S	3W	2S	
Lindane		8W		2W,3S	3S		5S		1W		
Toxaphene	2W										
Chlorobenzilate			4W	8W,3S	15W,3S	5W,3S	13W,7S	4W,3S	2S		
Dursban							1S				
Thiodan 1							2S				
Hydroxychlorden						1S					

36 samples/year (3 stations monthly)
W = water, S = sediments

water from the proposed S-319 drainage basin. These data, however, are more than 6 years old. The data collected at S-5AE, when flow was westward and the temporary plug in C-51 was in place, are more contemporary but represent only the western portion of the proposed drainage basin of S-319. Water quality for S-319 was estimated by averaging the data that were collected at S.R. 7 with the data that were collected at S-5AE. The data sets are described in Table B-1 and the final estimated water quality is shown in Supplement 1 of this appendix.

Based on the above averaging technique, the water at S-319 would be hard, highly mineralized and alkaline, with an estimated specific conductance of 890 micromhos/cm, calcium and magnesium levels of 87.6 and 10.7 mg/L, respectively, and alkalinity of 3.8 meq/L. The physical properties would include high color (143 Pt units) and low turbidity (7 NTU's). Total phosphorus should average 0.14 mg/L including 70% (0.1 mg/L) as dissolved inorganic ortho-phosphorus. Total nitrogen concentration should average 2.7 mg/L with 35% (0.9 mg/L) as inorganic forms.

Comparison of S-319 Water Quality to WCA-1 Interior Water Quality

Comparison of the estimated S-319 water quality to water quality inside WCA-1 was used to determine which water quality parameters would most likely be affected by the operation of S-319. The physical and hydrologic configuration of WCA-1 greatly affects its water quality and therefore complicates the assessment of water quality impact. The internal perimeter canal, which parallels the perimeter levees, first receives all surface inflows into WCA-1. The degree of interaction between the perimeter canal and interior marsh waters depends on the stage in WCA-1, local relief, and the quantity of surface inflows and outflows. These factors vary constantly throughout the year. Based upon an intensive SFWMD water quality investigation of the WCA's conducted between 1978 and 1981, a generalized distribution map was developed that delineates the areas that are influenced by surface inflows. This map (Figure B-2) was constructed by performing a K-means cluster analysis (BMDPKM) on the annual average total nitrogen, total phosphorus, and chloride concentrations at the 26 WCA-1 sampling sites. This cluster analysis showed three distinct zones which can be described as follows: a) The perimeter zone, located predominantly in the perimeter canal, had the highest nutrient and chloride levels. Since the surface inflows to WCA-1 also had high nutrient and chloride levels, these stations probably represent the area of WCA-1 that is most directly impacted by surface discharge. b) The interior zone had the lowest nutrient and chloride levels. Water quality at these stations was dissimilar to the surface inflows so this interior zone is probably the least affected by surface inflows. c) The transition zone had water quality that was better than the perimeter zone but poorer than the interior zone.

Estimated water quality at S-319 was compared to these three zones and to an averaged, area-wide concentration (Table B-6). This average was calculated by combining the water chemistry of that volume of WCA-1 that is represented by the perimeter canal (15%) with the water quality of the estimated volume of the WCA1 interior (85%). The volumetric average concentration for WCA-1 was calculated in order to derive a concentration for each water quality parameter that could serve as an area- wide estimate.

Average total and organic nitrogen concentrations for S-319 were lower than for WCA-1. The estimated pH value for S-319 was within one unit of the value for WCA-1. The dissolved oxygen concentration at S-319 would average 4.0 mg/L, which is about the same as the perimeter zone. No dissolved oxygen estimates are available for the transition and interior zones. The proposed S-319 pump station would therefore probably not cause an elevation in nitrogen levels, a change in pH levels, or a decrease in dissolved oxygen levels (excluding biological or chemical oxygen demands) in WCA-1.

The estimated average levels at S-319 for turbidity, ortho-phosphorus, total phosphorus, nitrate, ammonia, and calcium were greater than the values of these paramters for WCA-1. Operation of the proposed S-319 pump station may lead to an increase in levels of these parameters in WCA-1.

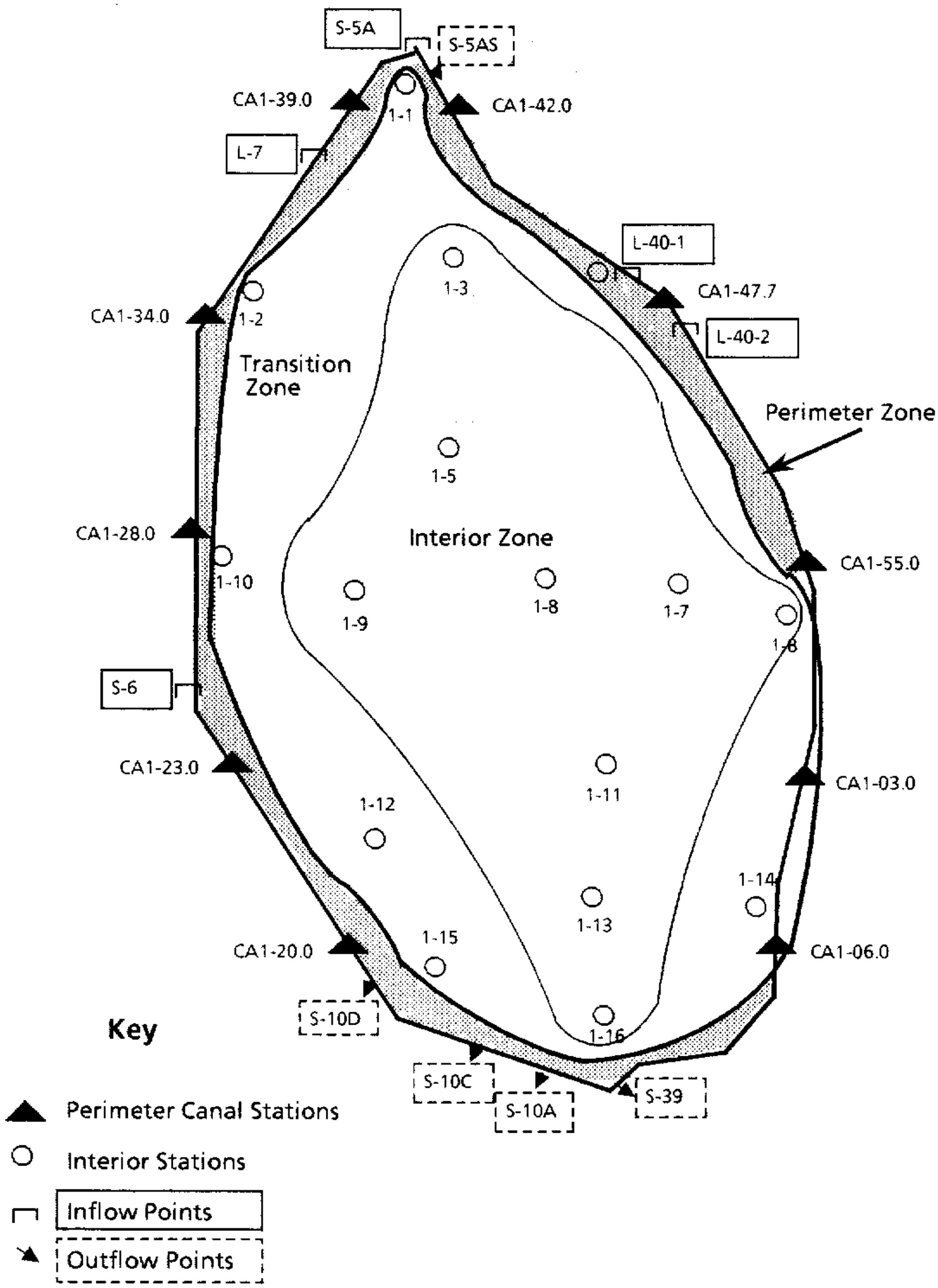

Figure B-2. Water Quality Monitoring Sites in Water Conservation Area 1 and Approximate Boundaries of Perimeter, Transition and Interior Zones.

TABLE B-6. COMPARISON OF ESTIMATED S-319 WATER QUALITY TO WCA-1 IMPACT ZONES

Parameter[1]	S-319 Estimate	Perimeter Zone	Transition Zone	Interior Zone	WCA-1 Average Volumetric Weight
Cond.(μmhos/cm)	890	1220	768	217	555
pH (units)	7.25	7.41	7.01	6.16	6.49
Turb. (NTU)	7.0	2.0	1.8	2.4	2.1
Color (units)	143	153	133	93	117
OPO_4	.097	.057	.011	.003	.012
TPO_4	.141	.099	.042	.024	.040
NO_2	.046	.067	.024	.005	.019
NO_3	.667	.567	.118	.007	.106
NH_4	.21	.16	.06	.11	.10
Org. N.	1.73	3.16	2.95	2.59	2.95
Total N	2.65	3.94	3.15	2.71	3.04
Ca	87.6	78.6	54.3	12.2	36.2
Mg	10.7	24.0	18.0	4.1	12.0
K	3.9	6.4	5.1	1.3	3.46
Na	57.4	111.3	82.2	22.8	57.1
Cl	111	162	119	38	85
Alk	3.76	4.79	3.53	.79	2.32
Hardness	263	295	210	47	184
SiO_2	7.9	17.0	15.9	6.3	13.1
SO_4	39.6	46.6	26.8	11.1	28.2
TOC	20.9	36.5	33.5	25.7	31.7
Cu	2.5	5.0	3.2	3.3	3.6

[1] All values in mg/L unless otherwise specified

The estimated average values for alkalinity, color, conductivity, and nitrite at S-319 were less than values of these parameters in the perimeter zone but greater than levels of these parameters in the remainder of WCA-1. Therefore, operation of S-319 probably would not result in increased levels of these parameters in the perimeter zone, but levels of these materials in the transition and interior zones may increase. Magnesium, potassium, and sodium levels for S-319 were greater than levels in the interior zone. For chloride, the level at S-319 was greater than both the interior zone and the volumetric average.

Comparison of S-319 Water Quality to Existing WCA-1 Inflows

Table B-7 presents a comparative ranking of water quality at existing inflow points to WCA-1 (S-5A, L-6, L-7, L40-1, and L40-2, Figure B-2) with the estimated water quality at S-319. S-319 water quality would have lower levels of specific conductance, organic nitrogen, total nitrogen, silicate, sulfate, sodium, potassium, magnesium, chloride, alkalinity, hardness, and total organic carbon. The quality of water from would also rank second lowest in nitrite, ammonia, and calcium. The pH, color, nitrate, and dissolved oxygen levels would fall in approximately the mid range of the existing inflows. The estimated concentrations of ortho and total phosphorus at S-319 would rank second highest, just below the levels at S-5A. The average turbidity of 7.0 NTU's, although low, would be approximately 3 to 5 times higher than turbidities of any existing inflows.

TABLE B-7. RANKING OF PROPOSED S-319 AND EXISTING WCA-1 INFLOWS

	D.O.	Conductivity	pH	Turbidity	Color	NO_2
High	L40-1 (5.3)	L7 (1978)	L40-1 (7.44)	**S319 (7.0)**	S5A (151)	S5A (.128)
	L40-2 (4.9)	S6 (1345)	S5A (7.37)	S5A (2.5)	L40-2 (147)	S6 (.108)
	S5A (4.1)	S5A (1306)	L40-2 (7.36)	L40-1 (2.4)	**S319 (143)**	L40-2 (.075)
	S319 (4.0)	L40-1 (1108)	**S319 (7.25)**	L40-2 (2.3)	L40-1 (141)	L40-1 (.057)
	S6 (3.4)	L40-2 (987)	S6 (7.24)	S6 (1.5)	S6 (112)	**S319 (.046)**
Low	L7 (1.3)	**S319 (890)**	L7 (7.16)	L7 (1.4)	L7 (105)	L7 (.035)

	NO_3	NH_4	Org N	Total N	OPO_4	TPO_4
High	S5A (1.269)	L7 (1.49)	S5A (3.57)	S5A (5.74)	S5A (.116)	S5A (.165)
	L40-2 (0.667)	S5A (0.75)	S6 (3.22)	L7 (4.73)	**S319 (.097)**	**S319 (.141)**
	S319 (0.667)	S6 (0.59)	L40-2 (3.15)	S6 (4.47)	S6 (.048)	L40-2 (.087)
	S6 (0.563)	L40-1 (0.36)	L7 (3.01)	L40-2 (4.08)	L7 (.047)	S6 (.078)
	L40-1 (0.454)	**S319 (0.21)**	L40-1 (2.91)	L40-1 (3.79)	L40-2 (.046)	L7 (.077)
Low	L7 (0.206)	L40-2 (0.18)	**S319 (1.73)**	**S319 (2.65)**	L40-1 (.030)	L40-1 (.066)

	SiO_2	SO_4	Na	K	Ca
High	L7 (32.2)	L7 (122)	L7 (301)	L7 (12.70)	S6 (95)
	S5A (21.9)	S5A (98)	S6 (138)	S5A (7.30)	L40-1 (94)
	S6 (21.1)	S6 (67)	S5A (130)	S6 (6.97)	S5A (93)
	L40-2 (15.0)	L40-2 (63)	L40-1 (122)	L40-2 (5.52)	L7 (92)
	L40-1 (13.8)	L40-1 (48)	L40-2 (102)	L40-1 (4.77)	**S319 (88)**
Low	**S319 (7.9)**	**S319 (40)**	**S319 (58)**	**S319 (3.90)**	L40-2 (80)

	Mg	Cl	Alk	Hard	TOC
High	L7 (43)	L7 (359)	L7 (7.11)	L7 (409)	S5A (42.0)
	S6 (35)	S5A (209)	S6 (6.52)	S6 (381)	L7 (38.7)
	S5A (31)	S6 (202)	S5A (5.49)	S5A (362)	S6 (38.5)
	L40-2 (20)	L40-1 (179)	L40-1 (4.93)	L40-1 (310)	L40-2 (35.3)
	L40-1 (18)	L40-2 (148)	L40-2 (4.70)	L40-2 (281)	L40-1 (33.8)
Low	**S319 (11)**	**S319 (111)**	**S319 (3.76)**	**S319 (263)**	**S319 (29.0)**

Effect of S-319 Pumpage on WCA-1 Nutrient Budgets

The preceding section presented qualitative comparisons between estimated water quality at S-319 and water quality within WCA-1. Such analyses provide a useful means by which individual parameters can be screened to determine their potential for altering the water quality of WCA-1. However, this comparative technique does not consider the quantitative loadings of these parameters that could be attributed to S-319.

The estimated levels of nitrogen and phosphorus at S-319 were combined with the annual volumes of water that would be pumped, as predicted by the hydrologic models for the various management configurations, to calculate the total mass loadings of nitrogen and phosphorus delivered to WCA-1 (Table B-8). The models also estimated the base flow into WCA-1, due to S-5A, S-6, and rainfall. Using the flow-weighted mean values for nitrogen and phosphorus concentration of each of these three inflows, base flow nutrient loadings were also calculated. Comparisons of the increase in water, nitrogen, and phosphorus loadings to WCA-1 due to operation of S-319, relative to the base flow levels, indicated that, on an annual

TABLE B-8. PREDICTED WATER, NITROGEN, AND PHOSPHORUS LOADS TO WCA-1 FROM S-319

	Base Run Inflows			S-155A = 1000 cfs			S-155A = 300 cfs		
Year	Water (Ac-ft)	N-Load (tonnes)	P-Load (tonnes)	Water (Ac-ft)	N-Load (Tonnes)	P-Load (Tonnes)	Water (Ac-ft)	N-Load (Tonnes)	P-Load (Tonnes)
1963	755,826	3,263	80	0	0	0	52,296	171	9.09
1964	1,260,404	5,441	134	4,217	14	0.73	79,819	261	13.88
1965	1,081,834	4,670	115	79,983	261	13.91	130,927	428	22.77
1966	1,633,751	7,053	173	36,954	121	6.43	163,766	535	28.51
1967	824,336	3,559	87	21,099	69	3.67	66,440	217	11.55
1968	1,452,765	6,272	154	138,910	454	24.16	199,689	653	34.93
1969	1,380,129	5,958	146	26,932	88	4.68	169,406	554	29.46
1970	1,115,925	4,817	118	28,965	94	4.99	129,922	425	22.59
1971	840,134	3,627	89	14,592	48	2.54	54,258	177	9.44
1972	915,107	3,950	97	40,615	133	7.06	121,568	397	21.14
1973	934,674	4,035	99	15,717	51	2.73	142,591	466	24.80
1974	1,063,375	4,591	113	22,448	73	3.90	99,562	325	17.31
1975	918,407	3,965	97	14,059	46	2.45	92,201	301	16.03
1976	905,057	3,907	96	4,704	15	0.82	108,051	353	18.79
1977	1,083,172	4,676	115	33,704	110	5.86	100,315	328	17.45
1978	1,178,225	5,086	125	17,737	58	3.08	126,692	414	22.03
1979	895,864	3,867	95	73,656	241	12.81	137,891	451	23.98
1980	887,722	3,832	94	7,021	23	1.22	72,513	237	12.61
1981	unavailable	-	-	36,495	119	6.35	68,417	224	11.90
AVG	1,062,595	4,587	113	32,502	106	5.65	112,577	368	19.58

basis, operation of S-319 could increase the phosphorus loadings to WCA-1 by up to 25% if S-155A were limited to a 300 CFS capacity (Figure B-3). In fact, the percent increase would be greater than 10% every year, would exceed 15% in 13 out of 18 years, and would exceed 20% in 5 of 18 years. The average increase for the period of record would be 18%.

If S-155A had a discharge capacity of 1000 cfs, the increase in phosphorus loadings would be significantly reduced relative to the 300 cfs structure. The maximum increase would be 16%, and the average would be 5%. Loadings would have been increased by less than 5% during 13 out of 18 years.

The percentage increase in phosphorus loadings into WCA-1, due to operation of S-319, are greater than the increases in loadings of water or nitrogen, since the estimated phosphorus levels at S-319 are greater than the concentrations of phosphorus in the existing inflows. For the 300 cfs structure, the maximum increase in nitrogen loading was 12% and the maximum increase in water flow was 15% due to operation of S-319. For the 1000 cfs structure, the maximum nitrogen and water increases were 7% and 9% respectively. In all but three years, the nitrogen loadings were increased by less than 5%.

FINDINGS

1. Water quality in the section of C-51 that would be pumped by the proposed S-319 pump station would be hard, highly mineralized, and alkaline with moderate levels of nitrogen and moderately high levels of phosphorus.

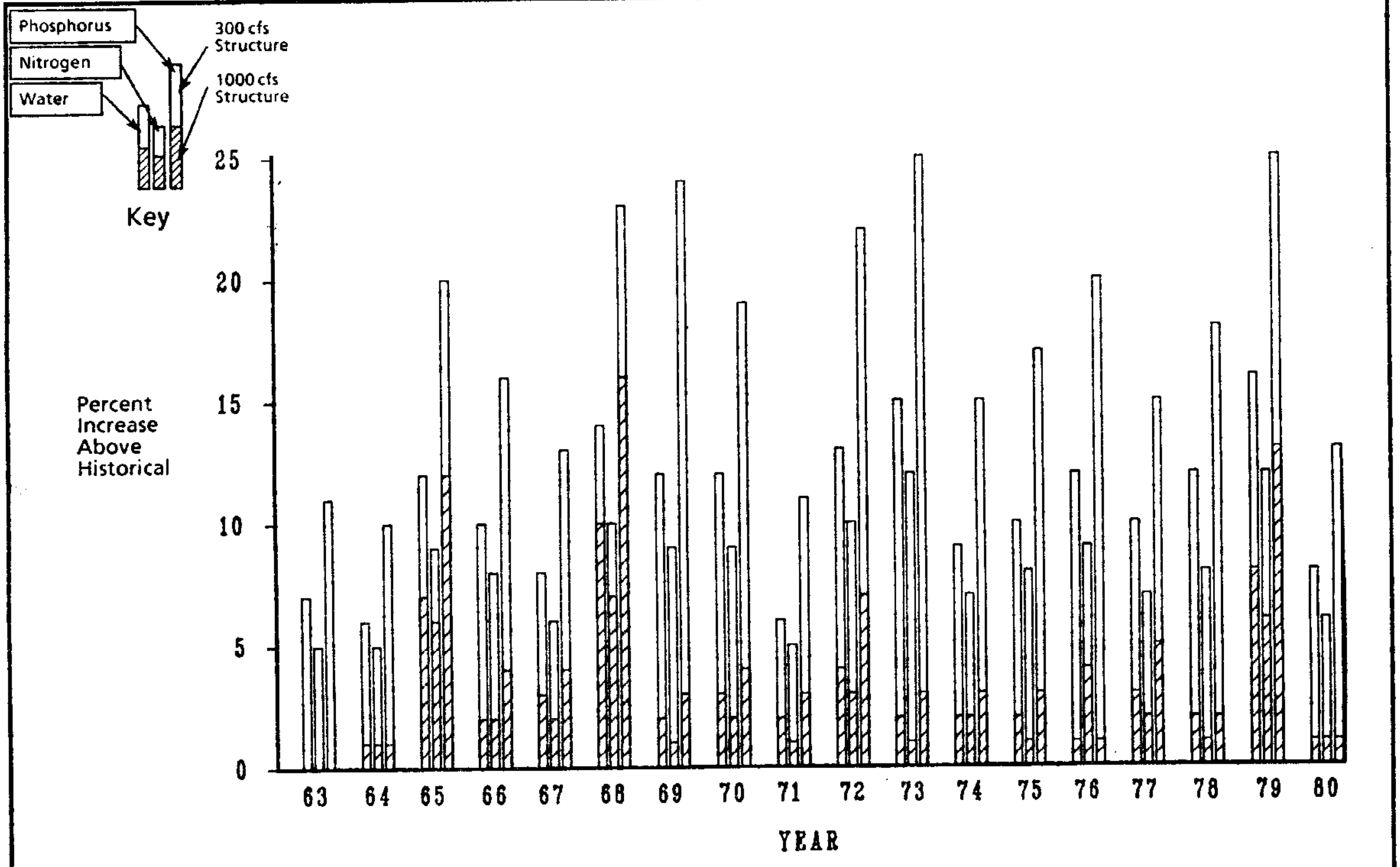

Figure B-3. Percentage Increase in Amounts of Water Nitrogen and Phosphorus Pumped into WCA-1 if the Proposed Pump Station S-319 were in Operation from 1963 to 1980. Shaded Area Represents Quantities Pumped with a Structure of 1000 cfs Eastward Discharge Capacity at S-155A. Unshaded Area Represents Quantities Pumped with a 300 cfs Capacity Structure at S-155A.

2. Compared to existing WCA-1 inflows, discharge waters from S-319 would be second from the highest in ortho- and total phosphorus. For all other parameters, S-319 would rank in the lower half, relative to other inflows.

3. Comparisons of the estimated S-319 water quality with the quality calculated for the various zones within WCA-1 indicate that for 15 of 22 parameters examined, S-319's water quality would at least be superior to the perimeter zone quality while for three parameters (total nitrogen, organic nitrogen and total carbon) the S-319 quality is superior to even the quality of the interior zone of WCA-1. Two of the seven parameters which have S-319 values greater than any of the WCA-1 zones are the dissolved inorganic nutrients (orthophosphorus and nitrate nitrogen), which are directly accessible to the aquatic plant communities.

4. Comparisons of existing hydraulic and nutrient loadings suggest that S-319, in combination with a 1000 cfs structure at S-155A, would slightly increase the loadings of water and nutrients into WCA-1. The average annual increases in water, phosphorus and nitrogen would be 3%, 5% and 2%, respectively, while the maximum annual increases would be 9%, 16% and 7%, respectively.

CONCLUSIONS

1. Past and current land use and associated agricultural practices may have adverse impacts on the water quality of the western C-51 basin. The use of pesticides and herbicides (especially in citrus groves) over the last 10 years, the practice of sewage sludge disposal on citrus groves and pastures, and the high levels of nitrogen that are associated with sugar cane cultivation on organic soils are all factors that could contribute to poor water quality in western C-51. Existing water quality in C-51 is generally better than that at other inflow points to WCA-1. Runoff water quality could be improved through the use of innovative stormwater management techniques in newly-developed suburban areas. Careful monitoring of water quality should be incorporated as an essential feature of a proposed backpumping plan.

2. Environmental studies by the SFWMD in WCA-1 suggest that plant communities can be significantly altered and degraded by nutrient enrichment and other changes in water quality. The quantitative relationships and ecological significance of these trends are not known. The areal extent of the impacts into WCA-1 will be a function of the rate of pumping and the water stage.

Since the nature and extent of any adverse impacts that these nutrients, or other pollutant loadings, may have on WCA-1 are unknown, it is advised that the loadings be minimized in the following ways: 1) minimize discharge volumes to the extent possible; 2) minimize frequency of pumping to the extent possible; and 3) minimize low stage, water supply backpumping.

3. For most water quality parameters, the S-319 discharges will have little or no impact due to their low levels. For orthophosphorus and nitrate nitrogen, possible adverse impacts will be avoided due to the low volumes of water discharged by S-319 based on 1000 cfs discharge to the east.

Appendix B-Supplement

SUPPLEMENT B-1. ESTIMATE OF WATER QUALITY AT PROPOSED S-319 PUMP STATION

Parameter[1]	S-5AE[2]		C-51 at SR7[3]		S-319 Estimate[4]	
Dissolved Oxygen	4.9	(3.2-6.8)	3.0	(1.7-5.1)	4.0	(1.7-6.8)
Cond. (Field)	953	(772-1598)	827	(320-1650)	890	(320-1650)
pH (units)	7.21	(7.03-7.58	7.28	(6.70-8.20)	7.25	(6.70-8.20)
Turb (NTU)	6.1	(2.6-17.9)	7.9	(3.1-14.0)	7.0	(2.6-17.9)
Color (units)	143	(45-217)		-	143	(45-217)
OPO_4	.108	(.006-.262)	.085	(.006-.341)	.097	(.006-.341)
TPO_4	.167	(.045-.318)	.114	(.025-.396)	.141	(.025-.396)
NO_2	.056	(.012-.107)	.036	(.004-.113)	.046	(.004-.113)
NO_3	.922	(.104-2.616)	.412	(.100-2.033)	.667	(.100-2.616)
NH_4	.21	(.06-.38)	.21	(.01-.47)	.21	(.01-.47)
Org N.	2.18	(.65-4.12)	1.28	(.60-2.15)	1.73	(.60-4.12)
Total N	3.37	(1.07-5.62)	1.92	(.79-4.68)	2.65	(.79-5.62)
Ca	93.4	(82.8-108.2)	81.8	(46.8-108.1)	87.6	(46.8-108.2)
Mg	12.5	(6.8-19.4)	8.9	(3.6-20.9)	10.7	(3.6-20.9)
K	4.2	(4.2-4.2)	3.6	(1.9-7.8)	3.9	(1.9-7.8)
Na	59.0	(59.0-59.0)	55.7	(20.0-99.3)	57.4	(20.0-99.3)
Cl	133	(91-304)	90	(4-149)	111	(4-304)
Alkalinity (meq/L)	3.81	(2.88-5.47)	3.70	(.10-6.07)	3.76	(.10-6.07)
Hardness	285	(235-350)	241	(132-336)	263	(132-350)
SiO_2	8.6	(7.0-11.3)	7.2	(4.6-10.5)	7.9	(4.6-11.3)
SO_4	34.4	(34.4-34.4)	44.8	(8.4-64.0)	39.6	(8.4-64.0)
TOC	20.9	(14.3-28.9)		-	20.9	(14.3-28.9)
Cu		-	2.5	(1.0-6.0)	2.5	(1.0-6.0)

[1]Units in mg/l unless otherwise noted. Phosphorus data presented as P

[2]During Flow to the West

[3]During Flow to the east with S-5A closed

[4]Average of S-5AE and C-51@SR7

WATER MANAGEMENT PLANNING FOR THE WESTERN C-51 BASIN

March 1984

Appendix C
SYSTEM ROUTINGS OF C-51 BASIN RUNOFF

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Appendix C

System Routings of C-51 Basin Runoff

I. Introduction

The routing model is a regional quantitative hydrological model that has been developed by the SFWMD to simulate water conditions in Lake Okeechobee, the Water Conservation Areas, and the Lower East Coast service areas. The model was designed as a management tool to study the effects of changes in policy, regulation schedules, structural features and future demands on regional water levels, and to forecast the behavior of the system under different climatic conditions. In the following sections, Parts II and III describe how the sources and sinks of water in the regional system are treated by the model. Part IV describes the principal components of the computer model itself, and Part V describes how the model was applied in the analysis of backpumping for the C-51 Basin.

II. Adjustments for Rainfall and Evapotranspiration

Because many options must often be tested rapidly over a wide range of climatological conditions, a quick response time is required of the model. This is accomplished by use of a simplistic mass balance, water budget approach for each of the water reservoirs. Daily storage changes, which are a function of rainfall and evapotranspiration (ET), are preprocessed for each reservoir. The volume of ET is proportional to the surface area of the storage area. Therefore, an adjustment is made to the storage values in the model before these values are added to the total storage of the appropriate reservoir at the beginning of each time step. These adjustments are generally based on actual historical rainfall and evapotranspiration conditions, although occasionally a percentage of normal rainfall is used in conjunction with the normal evapotranspiration. Seepage out of each reservoir is calculated based on the hydraulic head across the levees that enclose the reservoir.

III. Water Discharges

Once the effects of rainfall, evapotranspiration, and seepage are added to the storage of each region, the model determines if discharges between regions are required. Two types of water discharges are made from surface water reservoirs: 1) water use discharges, and 2) regulatory discharges. The discharges from one region to another are limited by the capacities of the structures and canals that interconnect these regions. These limitations are incorporated into the model in the form of stage-discharge breakpoint curves.

A. Water Use Discharges

Water use requirements are determined beforehand for each of the Lower East Coast and Lake Okeechobee service areas. Agricultural demands for the Lake Okeechobee service areas are defined based on the maximum estimated ET losses from the various crops that are grown in these areas. The estimated water use discharges are the proportion of these ET losses that must be supplied as irrigation water from Lake Okeechobee. Rainfall and local storage are the alternate sources for meeting ET requirements. Agricultural demands are defined mathematically by the following expression:

$$D = (PET * K - RF) * SA - LSA$$

where

D - is the agricultural demand (AF),
PET - is the potential evapotranspiration (FT),
K - is the crop K factor which is a function of crop density and type,
RF - is rainfall (FT),
SA - is area (acres), and
LSA - is local storage available (AF).

These demands are calculated on a daily basis. LSA is determined continually by the model as a function of rainfall and ET. A maximum value of local storage available is also estimated through a calibration process. Municipal demands of the region surrounding Lake Okeechobee, based on usage by those interests that obtain their water directly from the lake, are added to these agricultural demands.

The Lower East Coast service area demands are somewhat more complex to estimate due to the large transmissivity of the aquifer, the threat of salt water intrusion into coastal wellfields, and the large variety of land uses. An integrated canal-groundwater model is used to estimate the quantity of water that needs to be delivered to the service areas to maintain coastal canals at desired levels. During severe drought conditions, Local cutbacks may still be required at certain wellfields, depending on the proximity of these wellfields to the ocean and fresh water canals.

Each service area obtains water from one primary surface water reservoir. Lower East Coast service areas 1, 2, and 3 obtain their water supplies from Water Conservation Areas 1, 2A, and 3A, respectively. During dry periods, however, water levels may be too low to allow discharges to the service areas. At those times, the water needs of the Lower East Coast must be met by release of additional water from Lake Okeechobee. The routing model treats the calculated water demands as a sink, so that the amount of these demands is subtracted from the available storage of the appropriate reservoirs.

Limitations in the magnitude of deliveries are also incorporated into the model based on physical limitations of the structures, canals, and water availability.

B. Regulatory Releases

Lake Okeechobee and each of the WCA's have seasonal regulation schedules that determine the allowable levels of water in the reservoir. These schedules were designed to optimize the benefits of the reservoir for flood protection, water supply, and maintenance of natural ecosystems. When water levels exceed these regulation schedules, releases are made from the reservoir. When Lake Okeechobee is above regulation schedule, water may be released to the WCA's if these areas are below schedule, or through the St. Lucie Canal and the Caloosahatchee River to tidewater.

Water from WCA-1 may be released eastward through the West Palm Beach or Hillsboro canals. The amount of water that can be discharged via these routes may be limited, however, depending on water conditions in the coastal regions. The primary regulation releases from WCA-1 occur through the S-10 structures into WCA-2A.

Water from WCA-2A may also be released to the east through the Cypress Creek and North New River canals. These releases are again restricted by coastal water conditions. The primary regulatory discharges from WCA-2A occur through the S-11 structures to WCA-3A.

In WCA-3A, water may be released into Water Conservation Area 3B through S-151 or to Everglades National Park through the S-12 structures. WCA-3B, in turn, may release water through the Miami Canal to the Atlantic Ocean, except during periods when high water levels exist in the coastal basins. These regulatory water releases from one region into another are subtracted from or added to the storage of the appropriate reservoirs.

IV. Computer Model

A. Main Program

The computer model consists of a main program and ten subroutines or functions. SPWEB is the main program. The purpose of this program is to determine the computational order of the various calculations and procedures that are executed by the model. The main program calls up the specific subroutines in the appropriate time sequence. Figure 2 is a block diagram of the main program.

B. Subroutines

1. Subroutine INTDATA is called at the beginning of model execution to open input data files and to read the time invariant data into the program. These time invariant data include: a. The first and last year of the particular model run; b. Initial values of constants; c. Minimum storage levels for each reservoir; d. Stage-storage breakpoint curves for each storage area; and e. Stage-discharge breakpoint curves for the S-10, S-11, and S-12 structures. Three data files, DDATA, DDEL, AND DDMND are opened. DDATA contains the time invariant data. DDEL contains the preprocessed daily change in storage for each water reservoir and the daily regulation schedules. DDMND contains the preprocessed demands of each service area in the model.

2. Subroutine STATE compiles the effects of rainfall, ET, seepage, and other predetermined flow rates that are independent of the management options in this model.

3. Subroutine CTOSTG converts storage to stage using the breakpoint stage-storage curves for each area, and a linear interpolation scheme.

4. Subroutine CTOSTO converts Stage to Storage using the same breakpoint stage-storage curve and a similar linear interpolation scheme.

5. Function SEEP calculates seepage across a levee as a function of head potential, length of levee, and a seepage coefficient. The equation for this computation is:

SEEP = (HP * LEN * COEFF)

where

HP	- is the head potential, (FT)
LEN	- is the length of the levee (Mile)
COEFF	- is the seepage coefficient (CFS/FT Mile)

The calling arguments of the function include the stages on both sides of a given levee and a number that dictates where calculated values will be stored in the computer memory.

6. Subroutine TNSMX determines the maximum flow rates through major discharge structures. These discharge rates are calculated daily as a function of the stages on each day.

7. Subroutine DELIVER reads in the water use demands of the service areas each day and then estimates the amount of water available in each of the major surface water storage regions in the study area. When the amount of water in local storage is not adequate to meet the water requirements of a service area, this subroutine calculates deliveries from a surface water storage area that has the available water to the service area in need. Lake Okeechobee is the only source of water for the Lake Okeechobee service areas. The Lower East Coast service areas can obtain water from the Biscayne aquifer and the WCA's as their primary source of water, and from Lake Okeechobee as a backup source of water during periods when the WCA's are dry.

8. Subroutine REGLO estimates the quantity of regulatory releases that are made from Lake Okeechobee to the WCA's and to the coast through the St. Lucie Canal and the Caloosahatchee River. Water is delivered to the WCA's rather than through the St. Lucie Canal or the Caloosahatchee River, whenever possible, in an attempt to keep the water in the system for use during drier periods. If the WCA's are over schedule, water is then released through the St. Lucie Canal and the Caloosahatchee River.

9. Subroutine REGCA calculates regulation releases out of the water conservation areas. Primary releases are made through the S-10, S-11, and S-12 structures. If these releases are insufficient, additional releases may be made through coastal canals to the ocean. The amounts of water that are released to the coastal canals depend upon local conditions in these canals. This subroutine also ensures that Everglades National Park receives the monthly minimum supply of water that is required for environmental purposes.

10. Subroutine PRINT sums the daily discharges and prints stage and discharge sums on the seventh, fourteenth, twenty-first, and last day of each month.

V. Application of the model to C-51 Basin Runoff

The simulation of conditions in the regional system for the analysis of C-51 backpumping was based on the methods described above.

Historical Runoff. Estimation of historical runoff from the C-51 basin was determined based on the difference between the net basin outflow that passed through the West Palm Beach lock and the inflow that passed through the S-5A east structure. The portion of this runoff that originated in the western section of the basin was determined by the percentage of area covered by the western basin relative to the entire basin. These numbers were increased by 20% to account for larger runoff quantities that would occur due to future urbanization. Computed runoff values are presented in Table C-1.

TABLE C-1. Total Runoff (Acre-Feet) Generated from the Western C-51 Basin

	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
1963	3187	6657	5615	424	11623	6732	2757	5068	23727	23550	9247	8353	106940
1964	12813	10332	4003	4332	9047	18008	10715	11849	21573	21241	17697	10356	151966
1965	3941	4370	0	0	0	11022	29824	18105	10142	50801	39416	5526	173147
1966	16701	18514	10939	4427	9213	33624	41173	33337	16505	22354	6046	3297	216130
1967	3973	4919	1345	0	0	14916	18843	9685	13059	36961	10854	5324	119879
1968	1486	3354	4366	46	19188	83412	47759	21180	21789	40991	13051	1972	258594
1969	6712	2926	18669	4268	14329	28816	16665	20603	22862	38908	30760	14583	220101
1970	10578	14222	42022	15039	6524	25680	19319	16314	13357	15832	1771	179	180837
1971	450	214	0	0	2870	10340	14103	10913	18097	8755	30558	9342	105642
1972	8380	6085	2487	11094	39807	52940	20256	17221	11310	6127	9558	5353	190618
1973	3820	2579	2103	1166	3693	15348	37829	27317	38408	27537	8626	8426	176852
1974	26210	4752	1670	0	0	10568	20979	25694	20014	22935	10760	12083	155665
1975	3265	704	355	0	4060	17492	29538	8948	24423	29653	16342	1464	136244
1976	0	186	10251	0	10981	7359	22471	32815	32099	15259	2253	1317	134991
1977	4536	1135	688	67	13863	11302	6907	4034	54300	7581	3382	17425	125220
1978	21269	5048	5657	464	4346	20626	29023	17151	16497	21882	27156	12456	181575
1979	15654	3287	5217	10386	20039	5806	6026	9055	63182	29249	13282	8985	190168
1980	5982	9045	6280	5673	7936	10314	18111	13079	28525	7446	8263	3106	123760
1981	248	3328	1152	0	212	7797	5369	42011	22505	67	9051	264	92004
												Average	160017

Model Runs. Model runs were made with each of three different management options applied to the storm water runoff from the western C-51 basin. The first option was considered as the base run. In this model run, the system is operated on the basis of the present management policies, regulation schedules, and consumptive water use requirements , including the Interim Action Plan. The Interim Action Plan redirects water that historically was backpumped from the Everglades Agricultural Area to Lake Okeechobee and routes this water to the WCA's. The storm water runoff from the western C-51 basin is routed east through the West Palm Beach locks.

Option 2 is the same as Option 1, except with regard to management of the western C-51 basin storm runoff. Under this plan, the runoff from the western C-51 basin again is passed to the east, but through a new structure, S-155A, unless it exceeds the capacity of this structure (300 cfs). Once this capacity is exceeded, the S-155A gate is closed and all of the runoff from the western basin is backpumped through S-319 into WCA-1. Under these conditions, the full conveyance capacity of C-51 east of S-155A is needed for flood protection of the eastern basin.

Option 3 is the same as Option 2, but the capacity of S-155A is assumed to be 1,000 cfs.

Results. Plots of water stages in Water Conservation Areas 1, 2A, and 3A are presented in Figures 1, 2, and 3 respectively. These areas would be directly affected by C-51 backpumping. Under the present management schemes, stages in the WCA's differ substantially at times from the historical levels . Some of these differences are due to changes made to the regulation schedules for Lake Okeechobee and the WCA's; higher levels of water use by the urban and agricultural areas; and the Interim Action Plan . Other changes over the years include the addition of backpumping stations along the lower east coast, addition of the L-67 extension south of the S-12 structures, and the minimum required releases that are made to Everglades National Park. These differences are illustrated in Figures C-1, C-2, and C-3, (pages C-15 through C-23 at the end of this appendix) by comparing historical stages to those of the base run.

The effects of adding C-51 runoff to the WCA's are also illustrated in these figures. Quantities of water that would be backpumped with a 300 cfs eastward discharge structure at S-155A, and with a 1,000 cfs structure are presented in Tables C-2 and C-3 respectively.

The stage plots for WCA-1 indicate the simulated stages with base run conditions are lower than the historical stages during many spring months, due primarily to changes in the regulation schedule. However, during rainy times, simulated stages are often higher than historical stages due to the effects of the Interim Action Plan. Backpumping of C-51 tends to amplify the peak stages, as would be expected.

In WCA-2, simulated stages during the base run were usually much lower than historical stages due to the drawdown schedule.

In WCA-3, simulated stages vary from historical stages for a number of reasons including the Interim Action Plan, increased water consumption by coastal regions, and modifications of regulation schedules in other WCA's and Lake Okeechobee. However, when comparing the base run to the cases with C-51 backpumping, the changes are minimal. Bar graphs of S-10, S-12, and S-39 structure discharges appear in Figure C-4 (page C-24 of this appendix).

TABLE C-2. Volume of Runoff (Acre-feet) That Would be Backpumped to WCA-1 with a 300cfs Capacity Structure at S-155A

	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
1963	0	682	0	0	6129	0	0	1632	19970	18794	2632	2547	52386
1964	4270	6071	0	1250	4066	12831	670	8239	14325	14349	12280	16088	94439
1965	0	1745	0	0	0	1609	25684	9675	6619	46890	37405	1531	131158
1966	7008	14793	6815	0	7248	30147	39555	31313	8584	18591	0	0	164054
1967	0	2822	1162	0	0	12155	11445	3293	11512	34259	2571	0	79219
1968	0	964	809	0	16201	83412	44458	17933	15852	38480	4580	0	222689
1969	1674	716	12698	769	5831	26827	14320	15697	19490	36830	28546	6307	169705
1970	5441	5615	40632	11623	4756	23459	12926	10090	7722	7888	0	0	130152
1971	0	0	0	0	1281	2725	5145	1331	11375	0	27642	4856	54355
1972	0	0	607	5556	38224	52940	11183	6906	1206	0	5161	0	121783
1973	0	0	0	0	0	12843	37829	26210	38408	25359	0	2192	142841
1974	21670	0	0	0	0	656	13815	23798	11451	17449	5179	5720	99738
1975	0	0	0	0	1111	9529	26892	0	21259	25591	7982	0	92364
1976	0	0	2241	0	8830	3796	20252	32381	31087	8919	736	0	108242
1977	2442	0	0	0	12165	5407	0	793	53892	0	0	13155	87854
1978	17865	890	3116	0	2194	12768	25553	10977	10046	13031	21566	8910	126916
1979	10255	0	1567	7874	15090	2902	0	1962	61764	25178	8394	3152	138138
1980	5143	2844	2572	2340	4752	3830	12873	8097	24627	2188	3372	0	72638
1981	0	1051	0	0	0	2620	1474	40360	16699	0	6331	0	68535
												Average	113537

TABLE C-3. Volume of Water (Acre-Feet) That Would be Backpumped to WCA-1 with 1000 CFS Flow Capacity at S-155A

	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
1963	0	0	0	0	0	0	0	0	0	0	0	0	0
1964	0	0	0	0	0	2166	0	0	0	2059	0	0	4225
1965	0	0	0	0	0	0	15174	0	2668	36524	25758	0	80124
1966	2214	9604	0	0	0	5330	15616	4257	0	0	0	0	37021
1967	0	0	0	0	0	0	8497	0	0	12639	0	0	21136
1968	0	0	0	0	4415	79382	34021	0	2380	18958	0	0	139156
1969	0	0	0	0	0	4094	6524	0	0	9315	7047	0	26980
1970	0	0	26587	0	0	2158	0	0	0	0	0	0	28745
1971	0	0	0	0	0	0	0	0	0	0	14599	0	14599
1972	0	0	0	4348	8581	27757	0	0	0	0	0	0	40686
1973	0	0	0	0	0	0	4439	0	9283	2023	0	0	15745
1974	13178	0	0	0	0	0	0	0	0	9309	0	0	22487
1975	0	0	0	0	0	0	0	0	4641	9441	0	0	14082
1976	0	0	0	0	2309	0	0	2404	0	0	0	0	4713
1977	0	0	0	0	0	0	0	0	33763	0	0	0	33763
1978	2344	0	0	0	0	2202	1978	2118	0	0	9116	0	17767
1979	0	0	0	6165	0	0	0	0	53531	12091	1999	0	73786
1980	2333	0	0	0	0	0	0	0	4701	0	0	0	7034
1981	0	0	0	0	0	0	0	36560	0	0	0	0	36560
												Average	32559

Appendix C

SYSTEM ROUTINGS OF C-51 BASIN RUNOFF

Supplement 1

STRUCTURE DISCHARGE CAPACITY

Regulation Releases from Lake Okeechobee

To Water Conservation Area 1			Limited by:
	S-5A	1,600 cfs	HGS5
	S-6	400 cfs	Canal limitation
To Water Conservation Area 2A			
		1,600 cfs	Canal limitation
To Water Conservation Area 3A			
		2,000 cfs	Canal limitation
To St. Lucie Canal			
		1,000 cfs	Canal limitation
To Caloosahatchee River			
		9,300 cfs	Canal limitation

Supplement 1 - continued

RELEASES MADE FROM LAKE TO MEET DEMANDS
AND MAINTAIN STAGES IN CONSERVATION AREAS

To Water Conservation Area 1

Lake Stage	Structure	Daily	Limited by:
13.5	S-5A	1,600	HGS5
	S-6	400	Canal limitation
12.5	S-5A	1,600	Hillsborough Canal not
11.5	S-5A	800	available due to
10.5	S-5A	0	limitation of existing canal

To Water Conservation Area 2A - S-7

Lake Stage	Maximum Discharge (cfs)
10.0	475
10.5	680
11.0	845
11.5	1,010
12.0	1,175
13.0	1,200

To Water Conservation Area 3A - S-8

Lake Stage	Maximum Discharge (cfs)
10.0	600
10.5	860
11.5	1,280
12.0	1,480
12.5	1.660
13.0	1.840
13.5	2,000

Supplement 1 - continued

MAXIMUM RELEASES

From Water Conservation Area 1
to Water Conservation Area 2A through S-10

Stage WCA	Daily Discharge (cfs)
10.00	0
15.25	2,000
18.50	6,000

From Water Conservation Area 2A through S-11

Stage WCA-2A	Daily Discharge (cfs)
7.00	0
13.00	2,800
15.00	9,000
16.00	12,100

From Water Conservation Area 3A to to ENP through S-12

Stage WCA-2	Daily Discharge (cfs)
6.00	0
9.00	793
9.50	1,091
10.00	3,471
10.50	5,554
10.75	6,545
11.00	7,943
12.00	11,901

See Discharge Limitation Curves

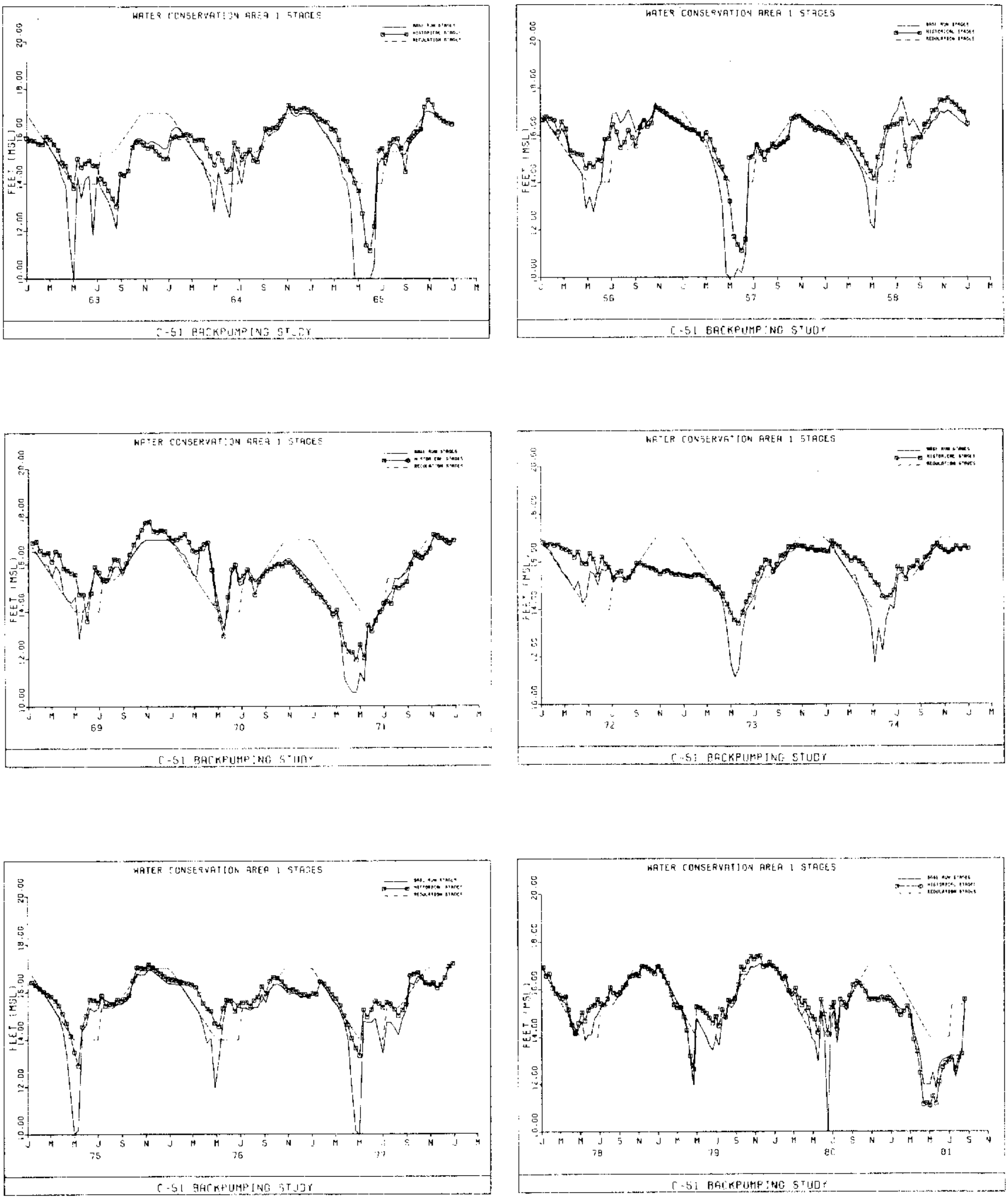

Figure C-1a WATER CONSERVATION AREA 1 STAGES UNDER BASE RUN AND HISTORICAL CONDITIONS IN RELATION TO THE REGULATION SCHEDULE. JANUARY, 1963 - SEPTEMBER, 1981.

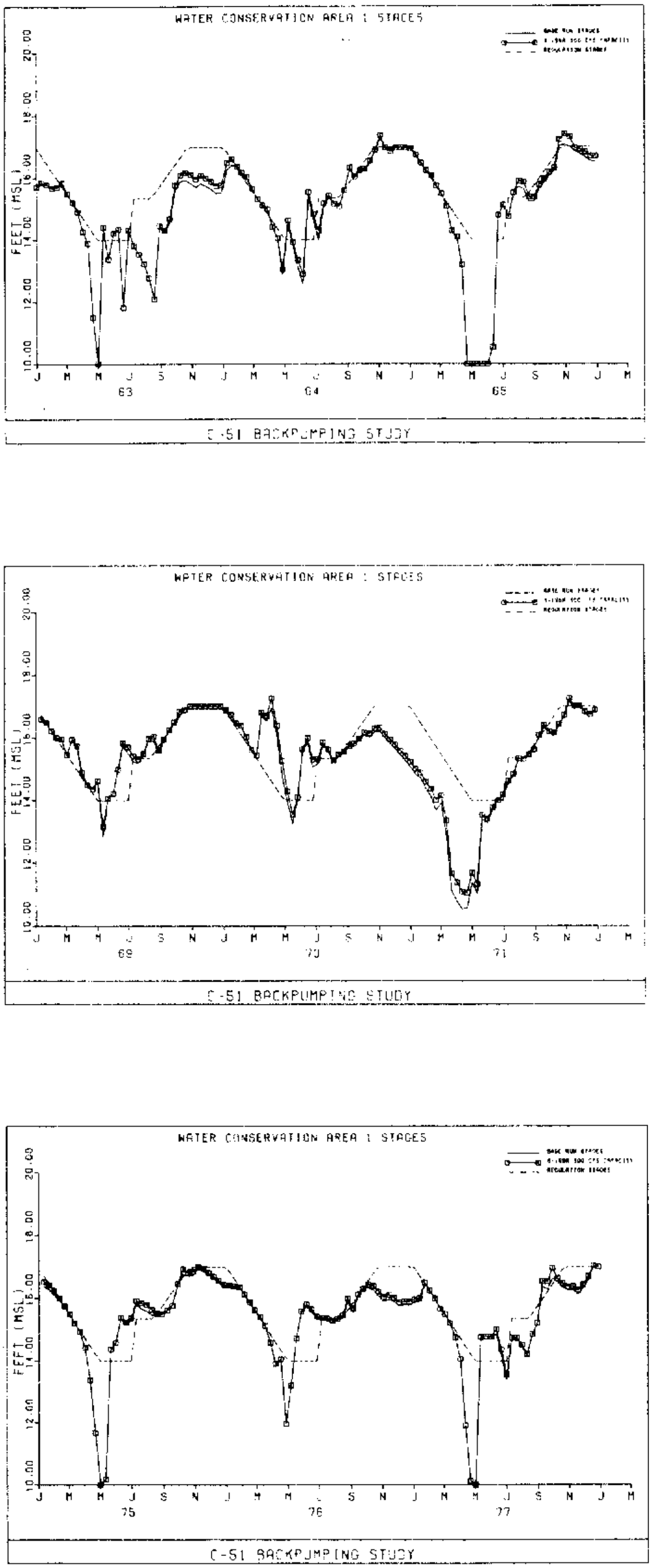

Figure C-1b **WATER CONSERVATION AREA 1 STAGES UNDER BASE RUN CONDITIONS;WITH BACKPUMPING AND A 300 cfs CAPACITY STRUCTURE AT S-155A IN RELATION TO THE REGULATION SCHEDULE. JANUARY, 1963 - SEPTEMBER, 1981.**

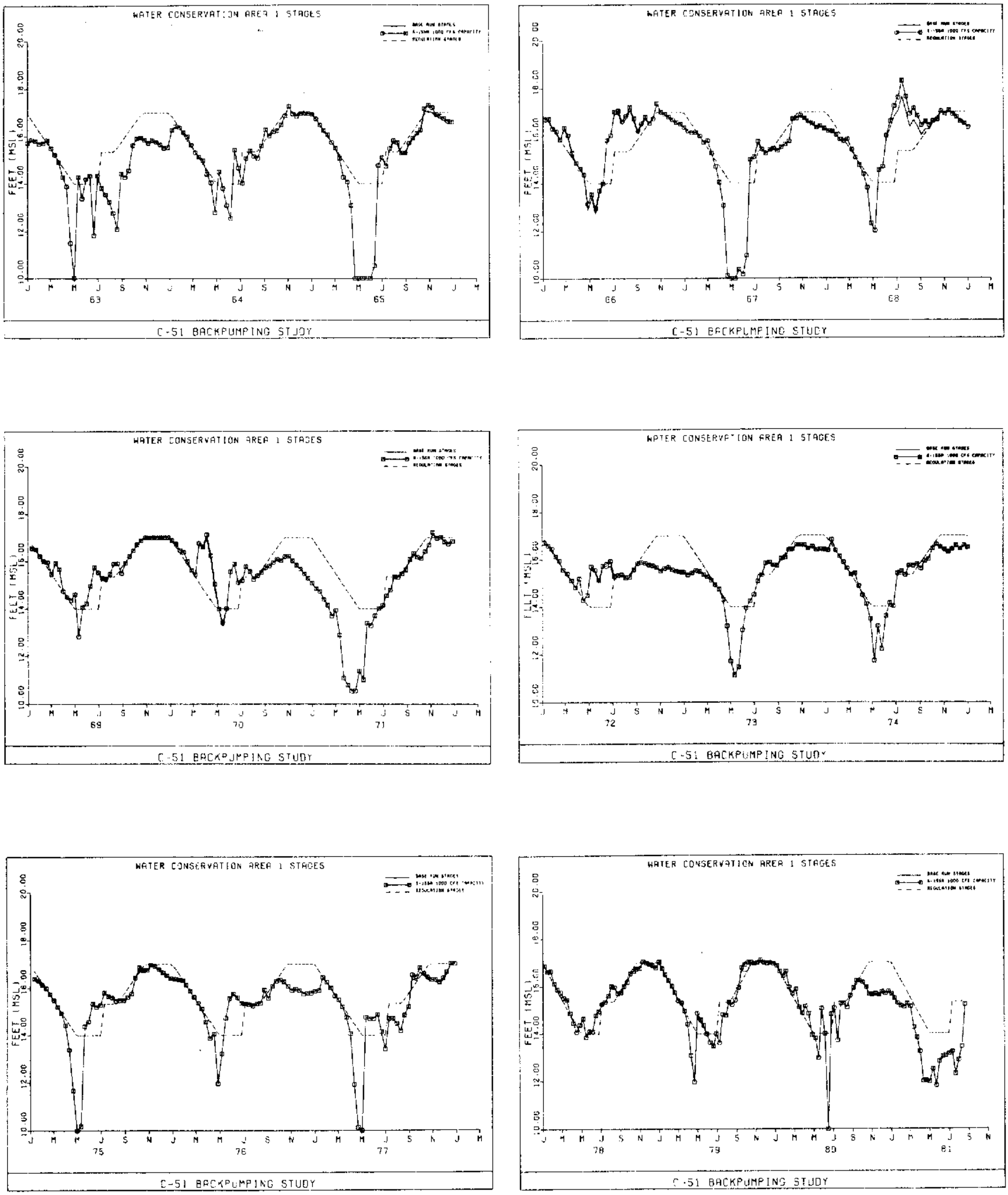

Figure C-1c **WATER CONSERVATION AREA 1 STAGES UNDER BASE RUN CONDITIONS; WITH BACKPUMPING AND A 1000 cfs CAPACITY DISCHARGE STRUCTURE AT S-155A IN RELATION TO THE REGULATION SCHEDULE. JANUARY 1963 - SEPTEMBER, 1981.**

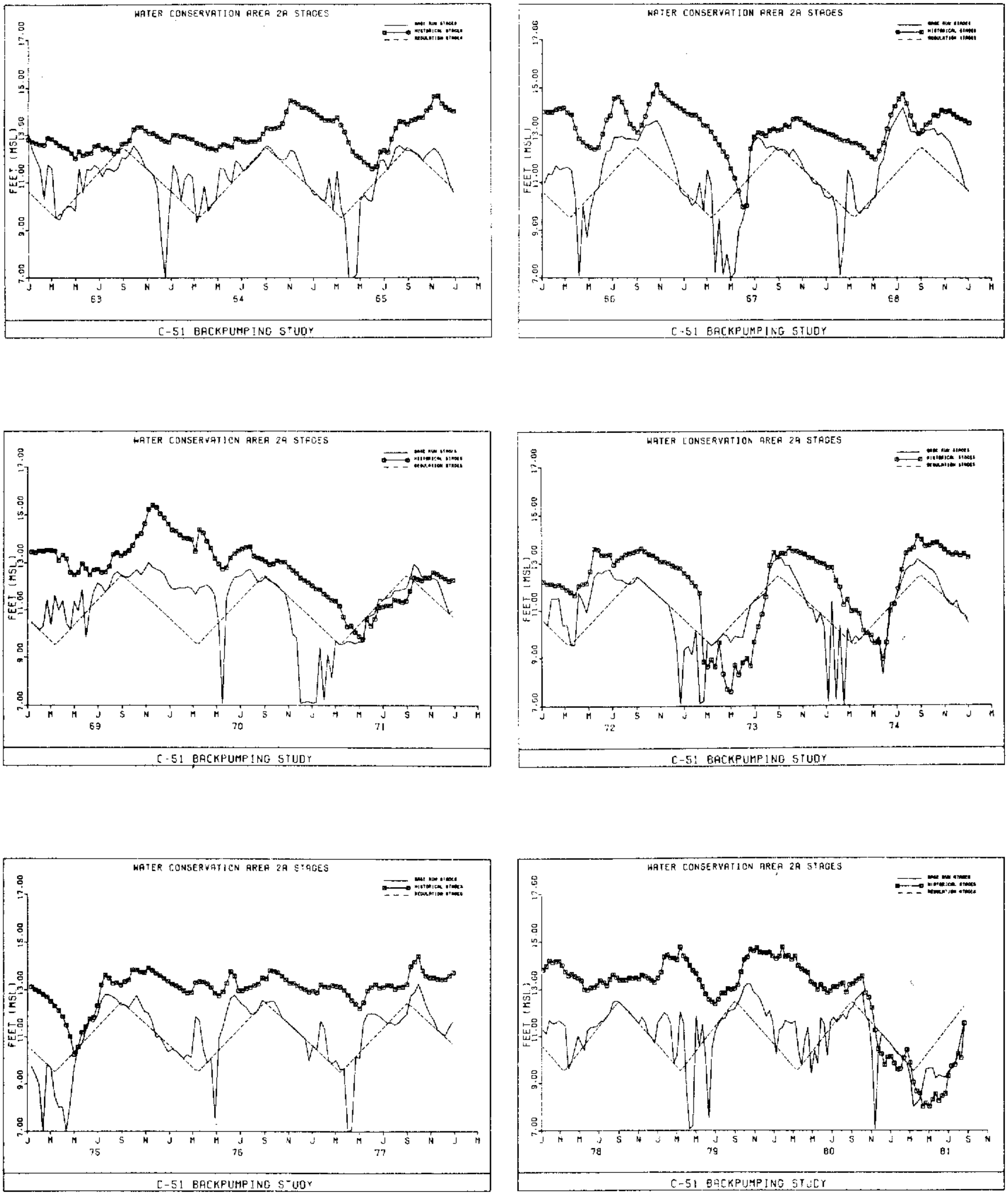

Figure C-2a WATER CONSERVATION AREA 2 STAGES UNDER BASE RUN AND HISTORICAL CONDITIONS IN RELATION TO THE REGULATION SCHEDULE. JANUARY 1963 - SEPTEMBER, 1981.

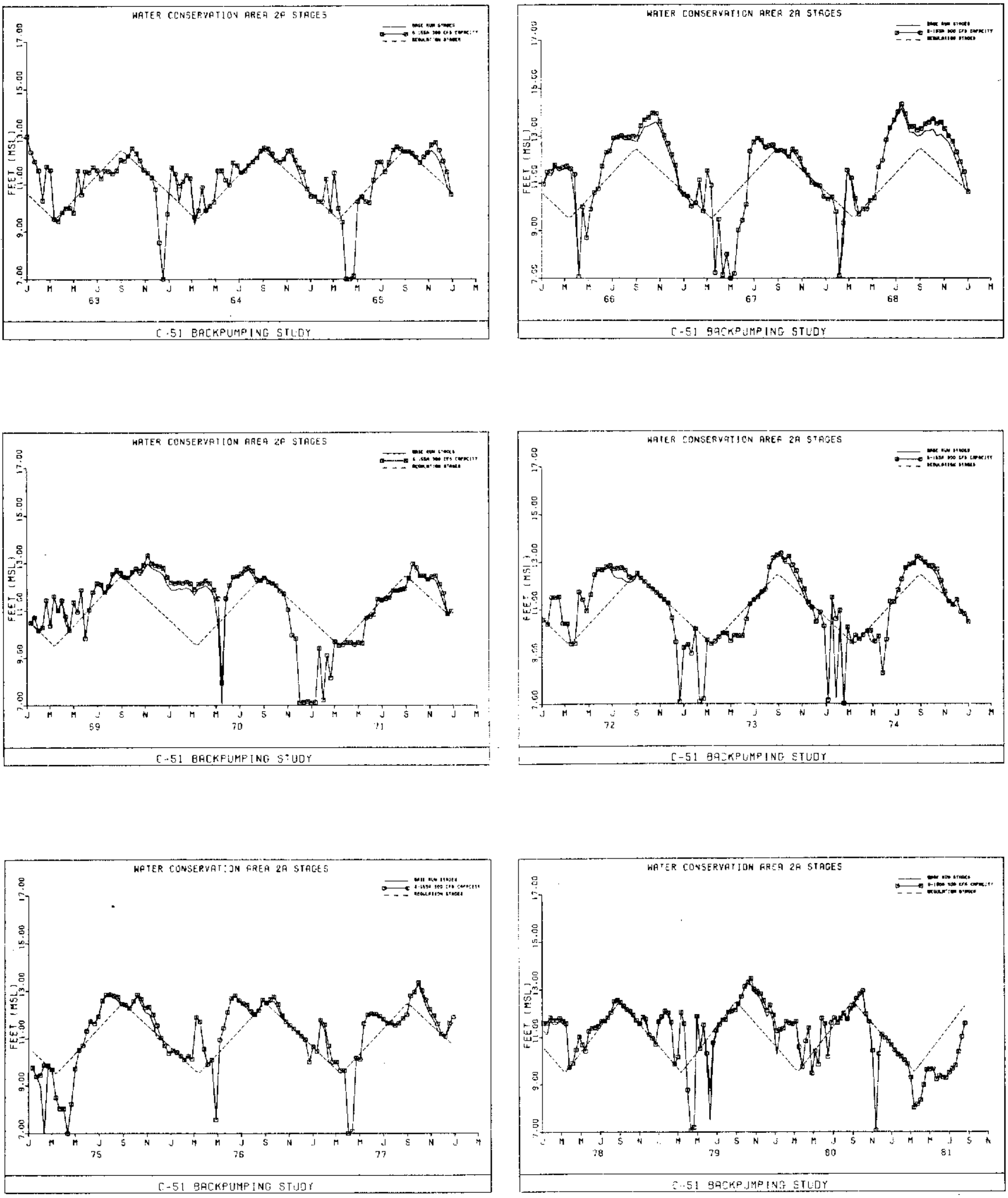

Figure C-2b **WATER CONSERVATION AREA 2 STAGES UNDER BASE RUN CONDITIONS; WITH BACKPUMPING AND A 300 cfs CAPACITY STRUCTURE AT S-155A IN RELATION TO THE REGULATION SCHEDULE. JANUARY 1963 - SEPTEMBER, 1981.**

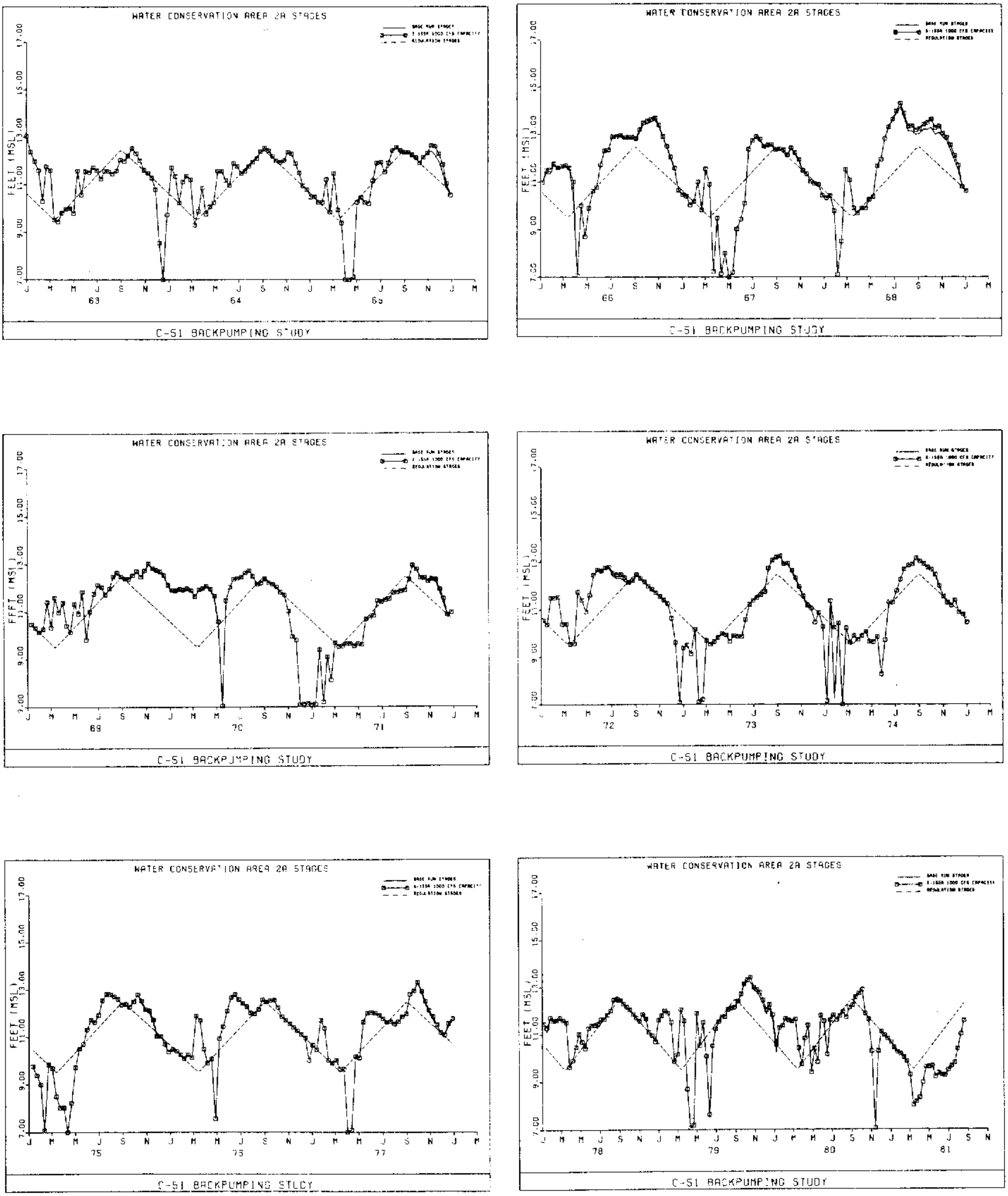

Figure C-2c WATER CONSERVATION AREA 2A STAGES UNDER BASE RUN CONDITIONS; WITH BACKPUMPING AND A 1000 cfs CAPACITY DISCHARGE STRUCTURE AT S-155A IN RELATION TO THE REGULATION SCHEDULE. JANUARY, 1963 - SEPTEMBER, 1981.

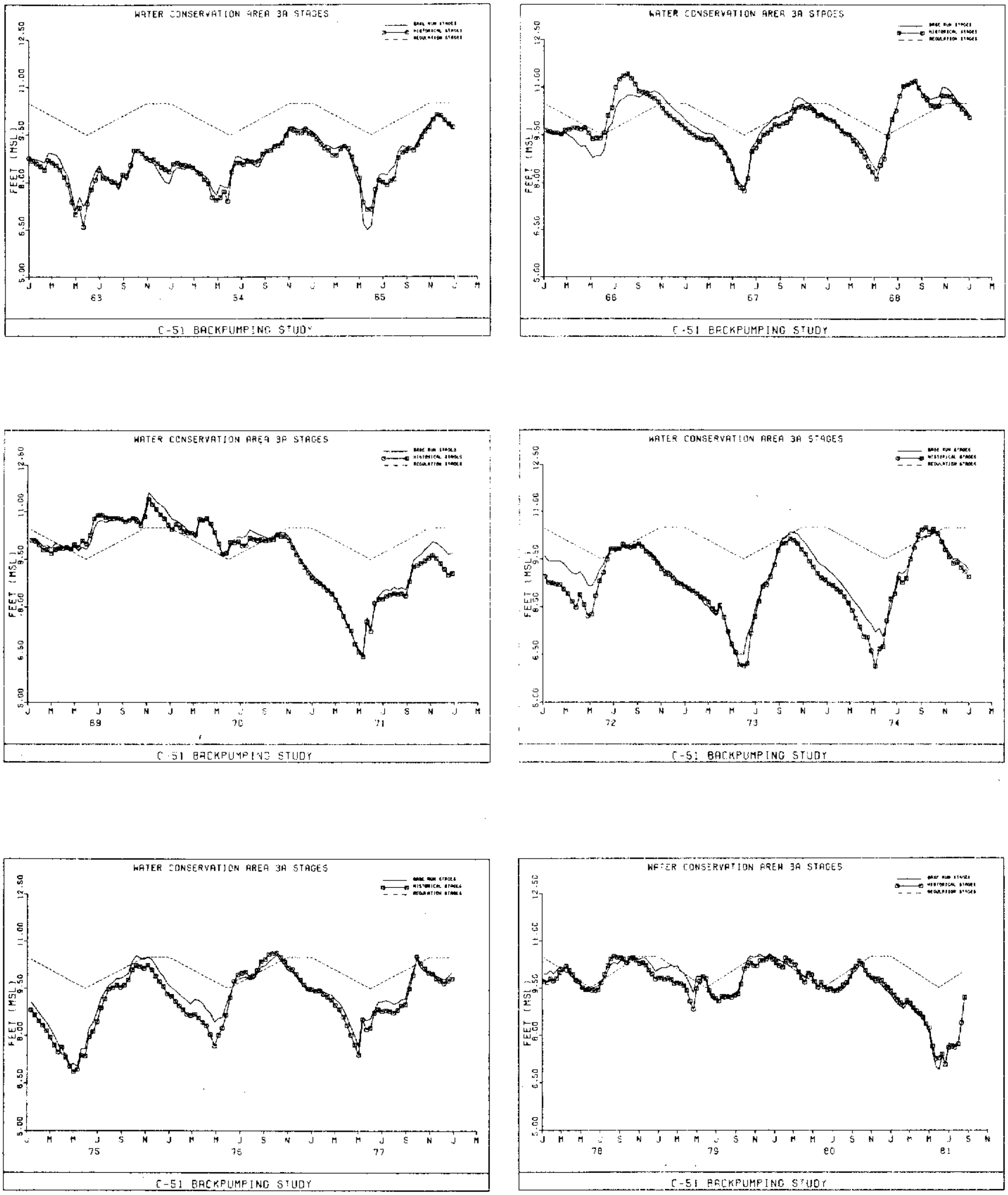

Figure C-3a **WATER CONSERVATION AREA 3A STAGES UNDER BASE RUN AND HISTORICAL CONDITIONS IN RELATION TO THE REGULATION SCHEDULE. JANUARY, 1963 - SEPTEMBER, 1981.**

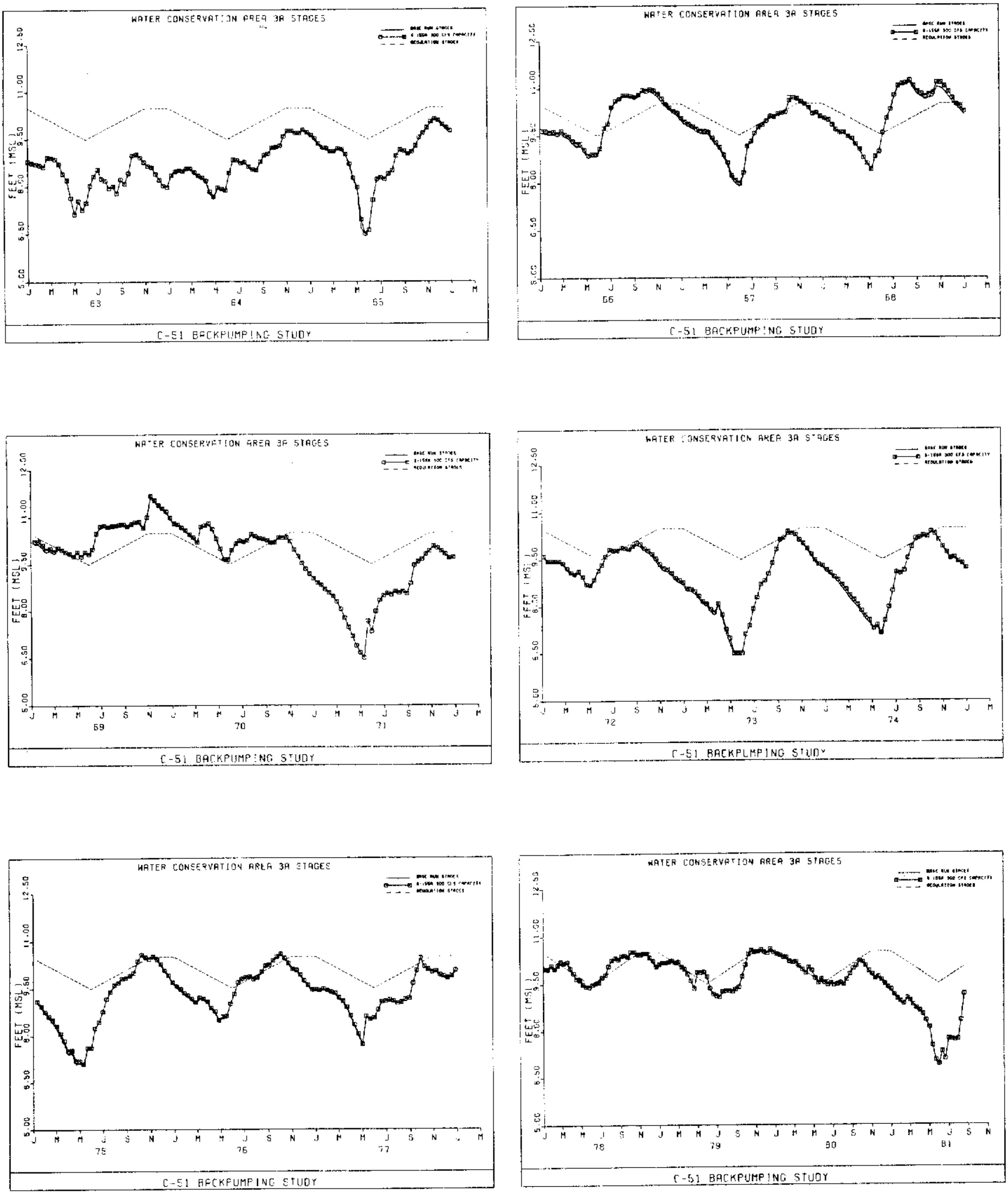

Figure C-3b **WATER CONSERVATION AREA 3A STAGES UNDER BASE RUN CONDITIONS; WITH BACKPUMPING AND A 300 cfs CAPACITY STRUCTURE AT S-155A IN RELATION TO THE REGULATION SCHEDULE. JANUARY, 1963 - SEPTEMBER, 1981.**

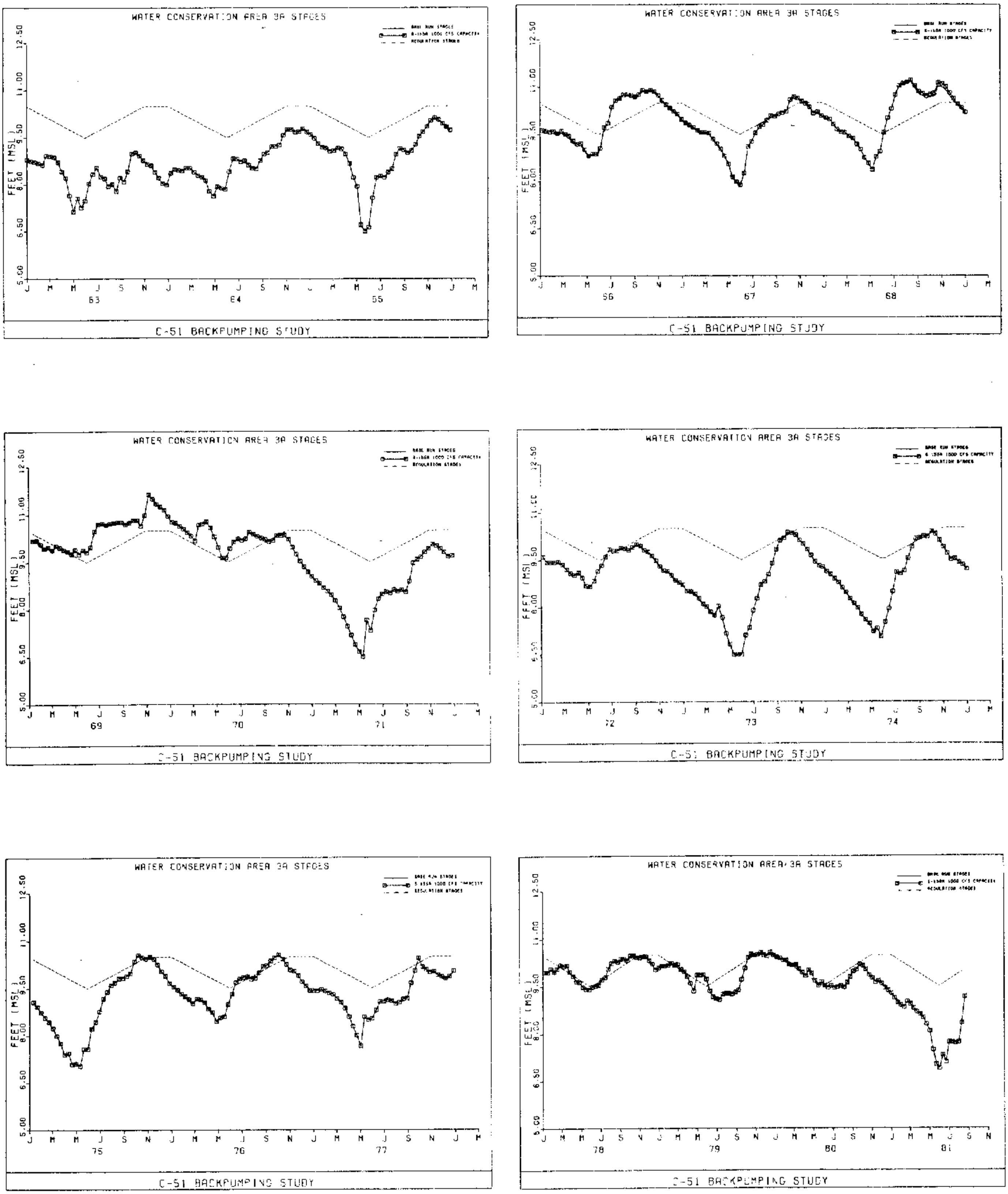

Figure C-3c WATER CONSERVATION AREA 3A STAGES UNDER BASE RUN CONDITIONS; WITH BACKPUMPING AND A 1000 cfs CAPACITY DISCHARGE STRUCTURE AT S-155A IN RELATION TO THE REGULATION SCHEDULE. JANUARY, 1963 - SEPTEMBER, 1981.

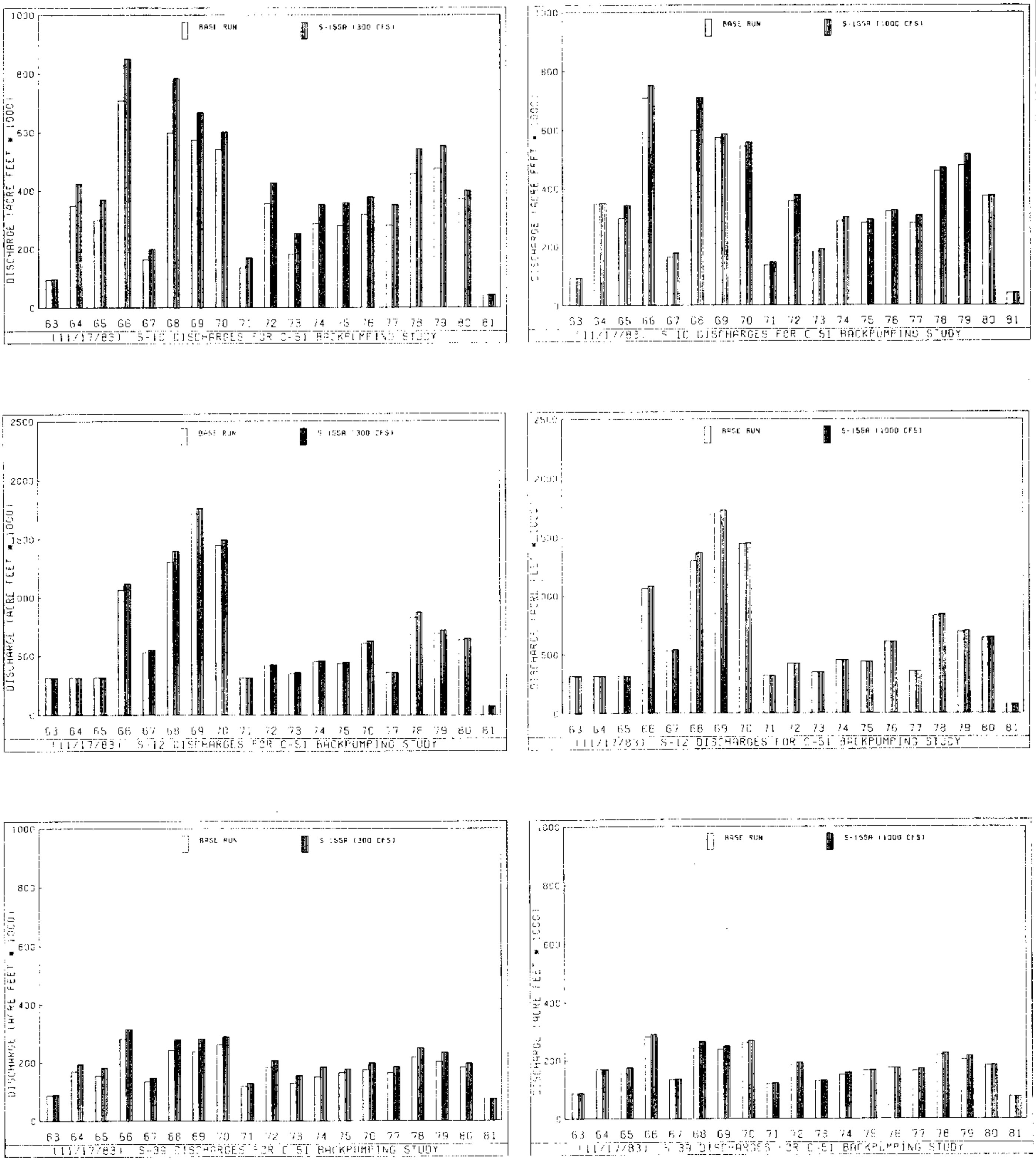

Figure C-4 **COMPARISONS OF DISCHARGES FROM THE S-10, S-12 AND S-39 STRUCTURES DURING BACK PUMPING WITH DISCHARGES UNDER THE BASE RUN CONDITIONS. GRAPHS ON THE LEFT ASSUME A 300 cfs CAPACITY EASTWARD DISCHARGE STRUCTURE AT S-155A GRAPHS ON THE RIGHT ASSUME A 1000 cfs CAPACITY STRUCTURE AT S-155A.**

WATER MANAGEMENT PLANNING FOR

THE WESTERN C-51 BASIN

March 1984

Appendix D
OCTOBER 22-24, 1983
STORM REPORT

Prepared by
Staff of the Resource Planning Department
South Florida Water Management District
West Palm Beach, Florida

HYDROMETEOROLOGICAL ANALYSIS

OF THE

OCTOBER 22-23, 1983 STORM

Water Resources Division
Resource Planning Department

Field Engineering Division
Resource Control Department

South Florida Water Management District
West Palm Beach, Florida

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I. Description

Heavy flooding was reported in Loxahatchee, Royal Palm Beach, Wellington, and West Jupiter after heavy rainfall poured down over the area during the night of October 22 and the morning of October 23, 1983.

Royal Palm Beach police chief, Noah Huddleston, reported that portions of Royal Palm Beach Blvd. were blocked off and other streets, including Meadowlark, Sparrows, Sandpiper, and South Swallow were covered with 1½ to 2 ft of water. Flooding also was reported in Boca Raton, where a section of a dike surrounding a sewage plant broke under pressure from the heavy rainfall and sent thousands of gallons of treated wastewater rushing toward homes in a residential area (Boca del Mar) west of the city.

According to a National Weather Bureau spokesman, the rain was due to a cold front which dipped into the southeast, moving north as its air masses warmed, creating the thunderstorms. The prevailing wind at Palm Beach International Airport was about 17 miles per hour in an easterly direction. The storm was over at about 1 p.m., October 23.

II. Rainfall Distribution

The rainfall, due to this frontal activity, began during the afternoon of October 22 continuing throughout the morning of October 23. Intensive rainfall occurred during the evening hours of October 22, and from 2 a.m. to 5 a.m. on October 23. Figure 1 presents the hourly rainfall distribution at four recording stations in the area: Timber Creek of Boca Raton, 1-8 Gage of Water Conservation Area 1, S-5A, and Jupiter Fire Station of Jupiter Farms. The spatial and time variations in rainfall intensity at these four locations are apparent. The rainfall occurred on and off during the days of October 22 and October 23 atTimber Creek with intensive rainfall during the hours of 2 a.m., 6 a.m.,and 8 a.m. of October 23. Two intensive rainfall periods were recorded at 1-8 Gage, and fairly uniformly distributed rainfall was recorded at S-5A and Jupiter Fire Station West.

Due to some rain gages not being read until Monday, October 24, the rainfall readings for this storm event were spread out from October 22 through October 24, 1983. Table 1 presents the daily rainfall values at available selected locations.

Western C-51 basin, between State Road 7 (SR-7) and S-5A, and the area in the vicinity of Pratt & Whitney, received the highest rainfall amount. A total of 8.80 inches was reported at Loxahatchee within a 24-hour period during the storm, and +?-14.0 inches was indicated by a gage located at the Wellington golf course. Since the gage overflowed, this reading was not included in Table 1.

West Palm Beach, Lake Worth, and Boca Raton received about 4 to 6 inches of rain during the storm. However, the area between the Florida Turnpike and Water Conservation Area 1 received slightly higher than 6 inches. The East Everglades Agricultural Area received about 2 to 4 inches of rain, and the West Everglades Agricultural Area received less than one inch of rain.

Figure 2 shows the isohyetal map of rainfall distribution resulting from this rainstorm in Palm Beach and Broward counties. Loxahatchee Grove, Wellington and portions of Royal Palm Beach received a total of 8 to 8.8 inches. This is about equivalent to a return frequency of 1 in 5year event. Eastern C-51 basin, especially the area east of Jog Road, received 4 to 6 inches, which is equivalent to a 1 in 2 to 1 in 3 year event, when compared with the rainstorm that occurred September 22-25, 1983 (see Figure 3). However, the Jupiter Farm area received much less rainfall than the last

TABLE 1. Daily Rainfall Values at Various Locations Prior and During the Rainstorm of Oct. 22-24, 1983

Station Name	10/17	10/18	10/19	10/20	10/21	10/22	10/23	10/24	10/25	Reading Time
Upper & Lower East Coast										
Ft. Pierce Field Station	1.25A	2.00	0.12	0.00	0.00	x	x	1.15A	0.00	7:25 am
Stuart	1.33	3.14	0.33	0.11	0.02	1.25	1.93	1.80	0.00	12 noon
St. Lucie	1.55	2.55	0.35	0.06	0.07	0.00	2.12	2.08	0.00	
Jupiter #1	1.40	2.72	1.10	0.00	0.00	0.00	2.20	0.95	T	8:00 am
Jupiter Fire Station	0.09	1/24	0.00	0.00	0.00	0.40	3.54	0.03	0.00	
Pratt & Whitney	1.70	2.40	0.00	0.00	0.15	5.00	4.00	0.10	0.00	Midnight
4434 Fuscia Ci S (P.B.Gardens)	0.28	0.75	0.09	0.05	0.13	0.24	5.32	0.13	0.00	6:00 pm
419 Sequoia Dr (W.P.B.)	0.70	2.10	0.25	0.09	0.00	0.00	2.85	0.78	0.10	7:30 am
City of West Palm Beach	0.64	1.05	0.04	0.00	0.04	0.00	2.77	0.17	-	8:00 am
Palm Beach Int'l Airport	0.73	0.48	0.07	0.00	T	0.99	2.87	0.32	0.00	Midnight
178 Drawdy Rd (W.P.B.)	0.20	2.81	x	0.27A	0.08	1.18	4.95	0.07	0.05	6:00 pm
S-5A	1.24	0.08	0.00	0.00	0.00	2.11	1.44	0.13	0.00	Midnight
4444 Regency Dr (W.P.B.)	0.06	1.25	0.30	0.00	0.08	0.01	3.76	0.36	0.22	8-10:30am
374 LaMancha Av (R.P.B.)	0.60	2.70	0.50	0.06	0.00	0.03	6.00	0.58		7:00 am
Loxahatchee	0.27	1.88	0.28	0.00	0.00	0.35	8.45	0.00	0.13	6:00 pm
Greenacres City	0.25A	1.30	0.20	0.01	0.10	x	x	5.13A	0.96	8-12 am
Lake Worth Rd & El Canal	0.60A	3.38	0.82	0.01	0.03	x	x	7.78A	0.62	8-12 am
Boynton Rd & Military Tr	0.80A	2.00	0.20	0.06	0.06	x	x	4.60A	0.10	8-12 am
Boynton Rd & E2	0.25A	2.20	0.50	0.02	0.00	x	x	5.68A	0.10	8-12 am
ot 30 & L.W. DD office	0.25A	1.50	0.13	0.01	0.45	x	L x	1.19A	0.15	8-12 am
Lot 28 & Rgeline	0.20A	2.00	0.32	0.02	0.05	x	x	6.40A	0.12	8-12 am
Delray Rd & E2	0.12A	2.22	0.12	0.01	0.52	x	x	2.74A	0.17	8-12 am
Lot 32 & Rgeline	0.38A	0.93	0.24	T	0.40	x	x	6.15A	0.21	8-12 am
Lot 38 & Military Trail	0.95A	0.36	0.05	0.00	0.00	x	x	1.17A	0.07	8-12 am
Lot 39 & Rgeline	0.14A	0.47	0.04	0.10	0.02	x	x	5.60A	0.05	8-12 am
Boca & Powerline Rd	0.30A	0.60	0.30	0.27	0.01	x	x	6.20A	0.05	8-12 am
Boca & Rageline Rd	0.15A	1.07	0.30	0.08	0.02	x	x	6.74A	0.05	8-12 am
Margate	0.12A	0.04	0.11	0.01	0.03	x	x	5.55A	0.03	8:00 am
Callery Judge Grove	-	-	-	-	-	x	+?-7.5	-	-	*
W.P.B. Field Station	0.65	0.44	0.00	0.00	0.00	0.82	3.04	0.35	0.00	Midnight
1-8 WCA 1 at C-40	0.21	1.03	0.02	0.00	0.00	3.05	2.51	0.00	0.00	Midnight

Table 1 - continued

Station Name	10/17	10/18	10/19	10/20	10/21	10/22	10/23	10/24	10/25	Reading Time
Sewell Lock	0.04	0.00	0.43	0.02	0.00	0.00	0.00	2.15	0.36	8:00 am
Ft. Lauderdale Field Sta.	0.23A	0.00	0.07	0.07	0.18	x	x	1.64A	0.73	6:30 am
Miami Field Station	0.13A	0.05	0.01	0.00	0.07	x	x	0.40A	0.08	7:00 am
Homestead Field Station	0.22A	0.69	0.01	0.00	0.00	x	x	0.04A	0.01	6:30 am
Pompano City Water Plant	0.03	2.45	0.00	0.07	0.00	1.74	1.39	0.02	0.01	Midnight
Gage 2-17 (WCA 2A)	-	-	-	-	-	-	3.47	0.47	0.02	
Gage 2-19 (WCA 2A)	1.11	0.06	-	-	-	-	3.13	T	-	
Everglades Agricultural Area										
Pelican #1	0.02	3.37	0.03	0.00	0.01	0.00	2.62	0.34	1.31	7-8:30am
Pelican #2	0.02	3.76	0.04	0.00	0.04	0.00	2.39	0.34	0.87	7-8:30am
Pahokee #1	0.00	1.26	0.00	0.05	0.00	0.00	2.34	0.10	0.05	7-8:30am
Pahokee #2	0.00	1.27	0.03	0.00	0.00	0.00	2.56	0.10	0.07	7-8:30am
East Shore	0.03	0.90	0.09	0.41	0.00	0.00	2.10	0.55	0.22	7-8:30am
S-7	0.05	1.09	0.00	0.00	0.00	x	x	3.75A		8:00 am
S-8	0.50A	1.07	0.25	0.00	0.00	x	x	0.10A	0.35	8:00 am
Belle Glade	0.00	0.32	0.00	0.55	0.00	0.00	1.67	0.04	0.20	8:00 am
Devils Garden	0.03	0.12	0.00	0.00	0.00	0.00	0.10	0.00	0.35	8:00 am
S-6	1.15A	0.55	0.00	0.00	0.00	x	x	2.55A	0.15	8:00 am

Notes: x = Accumulated on the following date
A = Accumulation
- = no information
T = Trace
* = Overflowed

event, while Loxahatchee and Wellington received almost twice as much as in the last event.

Table 2 shows the water levels in the Water Conservation Areas before and after this rainstorm. As a result of the rainstorm, the water level rose almost half a foot in WCA 1. There were no significant changes in WCA 2A and WCA 3A.

TABLE 2

Water Levels in Water Conservation Areas
Based on Daily Water Readings

Date	WCA 1	WCA 2	WCA 3
10/20/83	16.95	12.00	10.35
10/21/83	16.96	11.99	10.39
10/22/83	N/A	N/A	N/A
10/23/83	N/A	N/A	N/A
10/24/83	17.43	12.18	10.39
10/25/83	17.47	12.17	10.47

N/A - not available (Saturday and Sunday)

III. Antecedent Conditions

A. Rainfall. A significant amount of rainfall occurred in the C-51 basin and surrounding areas during the period of October 17 through October 22, 1983, as illustrated in Table 1. This rainfall created saturated or near saturated soil conditions in most areas of the C-51 basin.

B. Canal Stages Prior to the Storm. Since the greatest amount of rainfall occurred in West Palm Beach and the western C-51 basin, the water level conditions in those areas were examined. The following table presents the water levels on October 21, 1983, as compared to their optimum stages.

TABLE 3

Water Levels on October 21, 1983,
As Compared to Their Optimum States

Structure	Headwater Stage ft NGVD	Current Optimum Stage-ft NGVD	Normal Optimum Stage-ft NGVD
S-44	7.0- 7.2	7.0	6.8-7.3
S-41	7.0- 7.2	7.5	8.5
L-8 @ SR-441	13.5-14.2	12.0	12.0
Wellington Br.	11.7-11.8	7.5	8.5

The water levels were higher than normal at L-8 and Wellington Bridge. This was the result of thunderstorm activities in the area during the week of October 17, 1983.

The wet antecedent moisture conditions and higher than normal stage in L-8 and western C-51 canal did reduce available storage for the rainstorm of October 22-24, 1983.

OPERATION OF THE SYSTEM

I. S-44

This structure is a two-gated reinforced concrete spillway structure with automatic control. Its drainage basin (C-17) covers Riviera Beach, Lake Mangonia, the eastern portion of Palm Beach Gardens, Palm Beach Mall area, westward expansion area, and several square miles west of I-95 and south of 45th Street. This basin received about 3 to 6 inches of rainfall. The gates were on automatic until 6 a.m., October 23. Both gates were set at a 3.0 ft opening, then the gate openings were reset to 5.0 ft until 7 p.m., October 24. Figure 4 shows the hourly stage change at the headwater of S-44 during October 20 through October 27, 1983. General gate operations are also shown in the same graph. The system was operated properly.

II. S-41

This structure is a reinforced concrete gated spillway with two automatically controlled gates. The basin (C-16) covers the area of Boynton Beach, Lake Worth, Lantana, Hypoluxo, Atlantis, etc. The operation of the gates is pretty much similar to S-44. No problem was detected. Figure 5 shows hourly stage and gate operation during the period of October 20 through October 27, 1983.

III. Operation of C-51

There are three water control structures in the C-51 canal: S-5AE, at the extreme west end; G-124, about six miles to the east of S-5AE, and Palm Beach lock which discharges to Tidewater at the eastern end.

Palm Beach lock consists of two structures in parallel--a gated spillway with automatic control, and an eight barrel box culvert controlled by stop logs. The gate is operated automatically between 7.3 and 8.0 ft at headwater elevations. The stop logs are manipulated manually for larger discharges. There were five boards on the stop logs prior to this major rainstorm. All these boards were removed at 7 a.m., October 23, when the headwater elevation reached 7.89 ft. The headwater elevation was able to be maintained between 7.0 and 8.0 ft during October 23 throughout October 27. No problems were encountered in the eastern portion of the basin.

The stucture G-124 is a six barrel culvert with two barrels gated and four barrels with stop logs. Due to wet antecedent rainfall conditions in the basin, all gates were open full during the week of October 17, 1983 and throughout October 27, 1983. The staff gage at the upstream side of the structure was under water. The stage recorder at the Wellington Bridge, which is about 2 miles east of G-124, is shown in Figure 6. The water level rose from 11.1 ft NGVD at 2 p.m. of October 22 to 16.25 ft NGVD at 2 p.m. of

October 23, 1983. A flow measurement was taken at 10 a.m., October 23 at the Wellington Bridge. The total discharge was about 817 cfs at the stage of 16.0 ft NGVD.

There was flow through G-124 during the storm period, the stage was 16.35 ft NGVD at 11 a.m., October 23 and the upstream stage was about 0.2 ft higher. The peak stage at S-5AE during the storm period was 16.20 ft NGVD (Figure 7). The water level in C-51 peaked at the G-124 structure.

Structure S-5AE is a double barrel culvert which remained fully opened (7 ft) throughout the flood period. Flow was eastward from the L-8 canal until S-5AW was fully opened at 9 a.m., October 23. Runoff from the Everglades Agricultural Area, and L-8 basin was first pumped into Water Conservation Area 1 by pumping station S-5A. During the peak stage, the water in western C-51 canal flowed over its south bank and discharged into the Sucrose Grower area (see Field Inspection of Affected Areas).

The stage in western C-51 canal did not receded below 15.0 ft NGVD until 4 a.m., October 26. Figures 7, 8, and 9 present the hourly stage hydrographs and their gate operations at the S-5AE, L-8, and the S-5AW structure.

FIELD INSPECTION OF AFFECTED AREAS

I. <u>Royal Palm Beach</u>

Oct. 23, 1983

<u>Time: 10:28 a.m.</u>

Location: Amil Gates, M-1 Canal north of Southern Blvd.
Opened fully. The water was 1.0 ft from the top of the concrete bank on the upstream side and 1.8 inches from the top of the concrete bank on the downstream side.

<u>Time: 10:20 a.m.</u>

Location: "Old Section" of Royal Palm Beach, north of Southern Blvd. on Royal Palm Beach Blvd.
Swales were all full, some backyards were flooded, all canals full, some street flooding. Streets had about 0.75 feet of water in front of Greenway Village North building.

<u>Time: 10:27 a.m.</u>

Location: M-1 canal and Sparrow Drive. Water over bank.

<u>Time: 10:36 a.m.</u>

Location: Vicinity of Bob White Rd., south of Okeechobee Blvd.
Heavy street flooding on opposite side of bridge. Flooding approximately 2.0 to 2.5 ft from centerline of road. Water badly over bank.

<u>Time: 10:47 a.m.</u>

Location: Canal crossing south of Country Club Dr. and Royal Palm Beach Blvd.
Canal water over bank, water high in regard to bridge was about 2.5 ft in depth. Street flooding with 0.5 to 0.75 ft of standing water.

<u>Time: 10:52 a.m.</u>

Location: 130th St. and M-1 canal.
There was standing water in the yards of many homes along Orange Grove Blvd. The water at the upstream side of the culvert where the water was blocked by the bridge, was 1.25 ft from the top of the culvert. The water in the M-1 canal was near its bank.

<u>Time: 10:58 a.m.</u>

Location: Plug located just north of M Canal and M-1 Canal with 48 inch riser.

Some erosion was observed around the plug; however, no problem with the plug. Approximately 400 ft north from the plug, the water was at the top of the bank.

II. C-18 Basin

October 23, 1983

Time: 11:21 a.m.

Location: C-18 and Caloosa Weir.
Erosion on the north side of the Caloosa Weir was observed, it required immediate attention.

Time: 11:26 a.m.

Location: C-18 weir, north of Beeline Highway.
No problems were observed; the staff gage reading was 18.40 ft.

Time: 11:30 a.m.

Location: C-18 canal crossing PGA Blvd.
Except for minor street flooding, no significant problems were observed.

III. C-17 Basin

October 23, 1983

Time: 12:15 p.m.

Location: East of I-95, north of North Lake Blvd.
There were some streets with about 0.5 ft of standing water.

IV. Jupiter Farm Area

October 23, 1983

The Jupiter Farms area.
One or two streets under water in the south central portion. No visible flooding problems were found in the area.

V. Wellington

October 23, 1983

An on-site inspection revealed the following conditions:

Time: 11:15 a.m.

Location: Paddock Drive at C-10.
Canal overflowed with water approximately 0.30 ft on the center line of road.
Time: 11:20 a.m.

Location: Wellington Trace about 350 ft east of Paddock. Approximately 0.90 ft of water on the center line of road was observed.

Time: 11:25 a.m.

Location: Wellington Trace and C-3.
Approximately 0.80 ft water on centerline of road was observed.

Time: 11:30 a.m.

Location: First canal south of Wellington Trace on Greenview Shores. Approximately 0.40 ft of water on center line of road was observed.

Time: 11:31 a.m.

Location: Greenview Shores Blvd. and C-15.
Approximately 0.90 ft of water on the center line of road was observed.

Time: 11:34 a.m.

Location: Greenview Shores Blvd. and C-18.
Approximately 1.20 ft of water on the center line of road was observed.

Time: 11:37 a.m.

Location: Greenbrier Blvd. and C-4.
Approximately 1.0 ft of water on the center line of road was observed.

Time: 11:46 a.m.

Location: Greenbrier Blvd. and Wellington Trace. Approximately 0.90 ft of water on the center line of intersection was observed.

Time: 12:06 p.m.

Location: Wellington Trace and C-18.
Approximately 0.20 ft of water on center line of road was observed.

Time: 12:23 p.m.

Location: First canal east of Greenview Shores on Wellington Trace. Approximately 0.90 ft of water on the center line of the road was observed.

Time: 12:32 p.m.

Location: Big Blue Trace and C-15.
Approximately 0.40 ft of water on the center line of the road was observed.

Time: 12:40 p.m.

Location: Big Blue Trace and C-12.
Approximately 0.90 ft of water on the center line of the road was observed.

Time: 12:41 p.m.

Location: Wellington Elementary School.
Dry, little standing water, no significant problems were observed.

Time: 12:47 p.m.

Location: Forest Hill Blvd. and First Canal south of C-51.
Standing water on Forest Hill Blvd was observed.

Time: 12:53 p.m.

Location: Forest Hill Blvd. and C-13.
Approximately 1.1 ft of water on the center line of the road was detected.

Time: 1:00 p.m.

Location: Forest Hill Blvd. and C-17A (northbound lane).
Approximately 0.30 ft of water on the center line of the road was observed.

Time: 1:04 p.m.

Location: South Shore Blvd. and First Canal south of Forest Hill Blvd.
Approximately 0.70 ft of water on the center line of the road was encountered.

Time: 1:10 p.m.

Location: South Shore Blvd. and C-4.
Approximately 0.90 ft of water on the center line of the road was observed.

Time: 1:18 p.m.

Location: Impoundment area for country place. Area appeared to be okay. Water level was approximately 3 ft from the top of the levee. No homes or businesses appeared to be in jeopardy.

VI. **Homeland**

Although flooding observed, no houses were affected.

VII. Rustic Ranches

Homesites had standing water which is normally the case in this area during heavy rains. No homes were threatened.

VIII. Sucrose Growers

The stage in West Palm Beach Canal was high and overflowed its bank at a spot approximately 0.9 of a mile east of S-5A. The area of the "overflow" was approximately 150 ft wide with a depth of 4 to 6 inches over the south bank of C-51 canal. This overflow began to flow into the forebay of the Sucrose discharge pump. The forebay began to fill and finally began to flow over the approximately 200 ft. of the access road and into the cane field.

SUMMARY OF RAINFALL EVENT

This rainstorm was caused by a cold front which dipped into the Southeast, moving north as its air masses warmed, creating the thunderstorm. The prevailing wind at Palm Beach International Airport was about 17 miles per hour in an easterly direction. The major rainfall began about noon, October 22, 1983, and was generally over by noon, October 23.

A total rainfall of 6 to 8.8 inches was recorded over the western C-51 basin in the 24-hour period. Loxahatchee recorded 8.8 inches and Pratt & Whitney recorded a total rainfall of 9.10 inches. Eastern C-51 basin, especially the area east of the Florida Turnpike received between 3.5 to 6 inches of rainfall. Rainfall received in the area of Wellington, Loxahatchee, and portions of Royal Palm Beach approached a 1 in 10 year return frequency.

As a result of this rainstorm, portions of Royal Palm Beach Blvd. were blocked off and several other streets in Royal Palm Beach, such as Meadowlark, Sparrow, Sandpiper and South Swallow were covered with 1½ to 2 ft of water. Several streets in Wellington were also flooded resulting from an overbank flow from their canal system. These streets, including Paddock Drive at C-10, Wellington Trace, Greenview Shores Blvd., Greenbrier Blvd., Big Blue Trace, Forest Hill Blvd. at C13 and C-17A, and South Shore Blvd., were covered by 0.3 to 1.1 ft of water on the center line of the road. The stage in C-51 canal near S-5AE was high and overflowed its south bank at an area approximately 0.9 miles east of S-5AE. The area of the overflow was approximately 150 ft wide with a depth of 4 to 6 inches over the south bank of C-51 canal and into Sucrose Growers sugarcane field, but no houses were reported flooded.

Review of the operational data at the S-44, S-41, Palm Beach lock, and S-5A complexes indicated that the system was operated properly. Several points are worthwhile to mention as a result of this rainstorm:

1. Only approximately one-third of western C-51 basin received close to 1 in 10 year storm event during this storm event. The rest of the western C- 51 basin received 4 to 8 inches which is between a 1 in 2 year and a 1 in 5 year event. The stage in the West Palm Beach Canal would have been higher if the entire C-51 basin had been subjected to a a 1 in 10 year event.
2. The canal stage in L-8 reached 15.83 ft NGVD during this rainstorm. If pumping station S-5A was not operated during this rainstorm to relieve the flood stage in L-8 and the western C-51 canal by opening the structure S-5AW, or had a heavy rainfall also occurred in the L-8 basin and the S-5A basin, the flood stage in western C-51 and L-8 would have been considerably higher with significant flooding. A drainage plan to protect the L-8 basin from flood damage should be considered.
3. The two amil gates at M-1 canal in Royal Palm Beach, were opened full, and water was backing up in several lateral canals of the older section of Royal Palm Beach due to the M-1 canal not being able to handle any additional water. The runoff generated from the newly developed area north of Okeechobee Blvd. and the Royal Palm Beach Acreage area did contribute to this additional water. The M-1 project for Royal Palm Beach Acreage should be implemented as soon as possible.
4. The flooding in Wellington was due to lower road pad elevation (below 17.50 ft NGVD). Some areas were even below 15.0 ft NGVD.

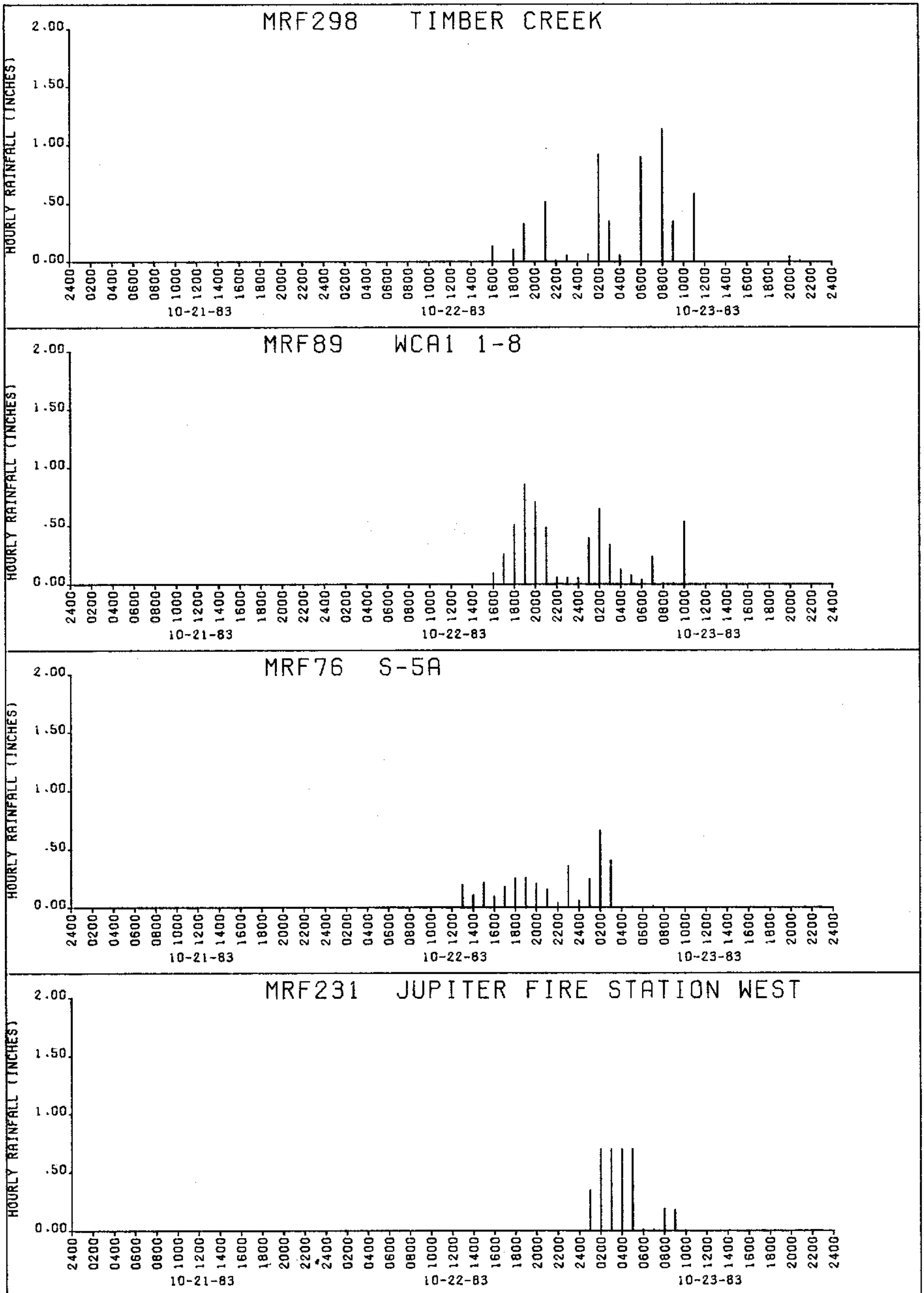

FIG. 1 RAINFALL EVENT 10/20/83/-10/23/83

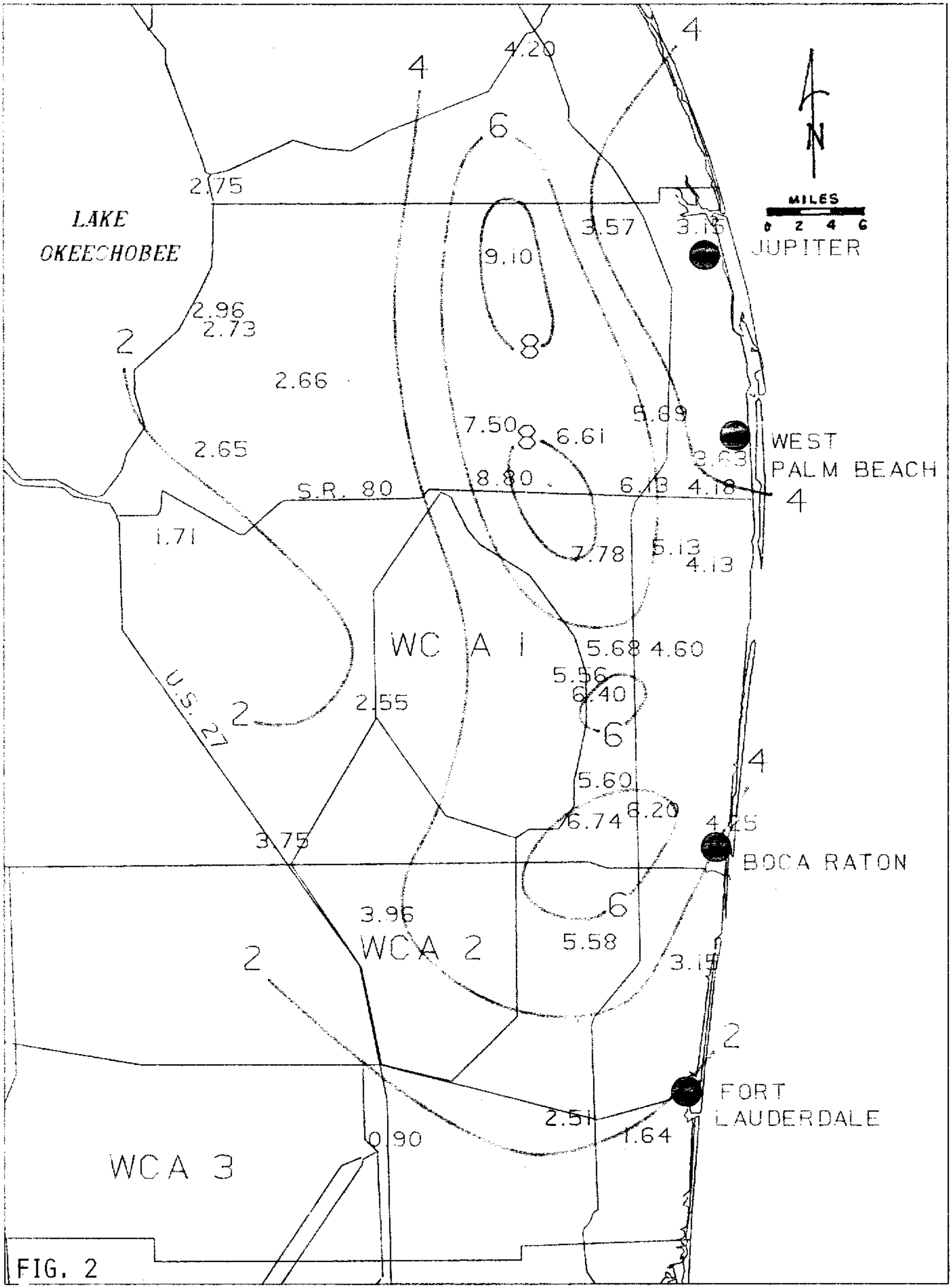

FIG. 2

ISOHYETAL MAP OF OCTOBER 22-24, 1983 STORM IN MARTIN, PALM BEACH AND BROWARD COUNTIES

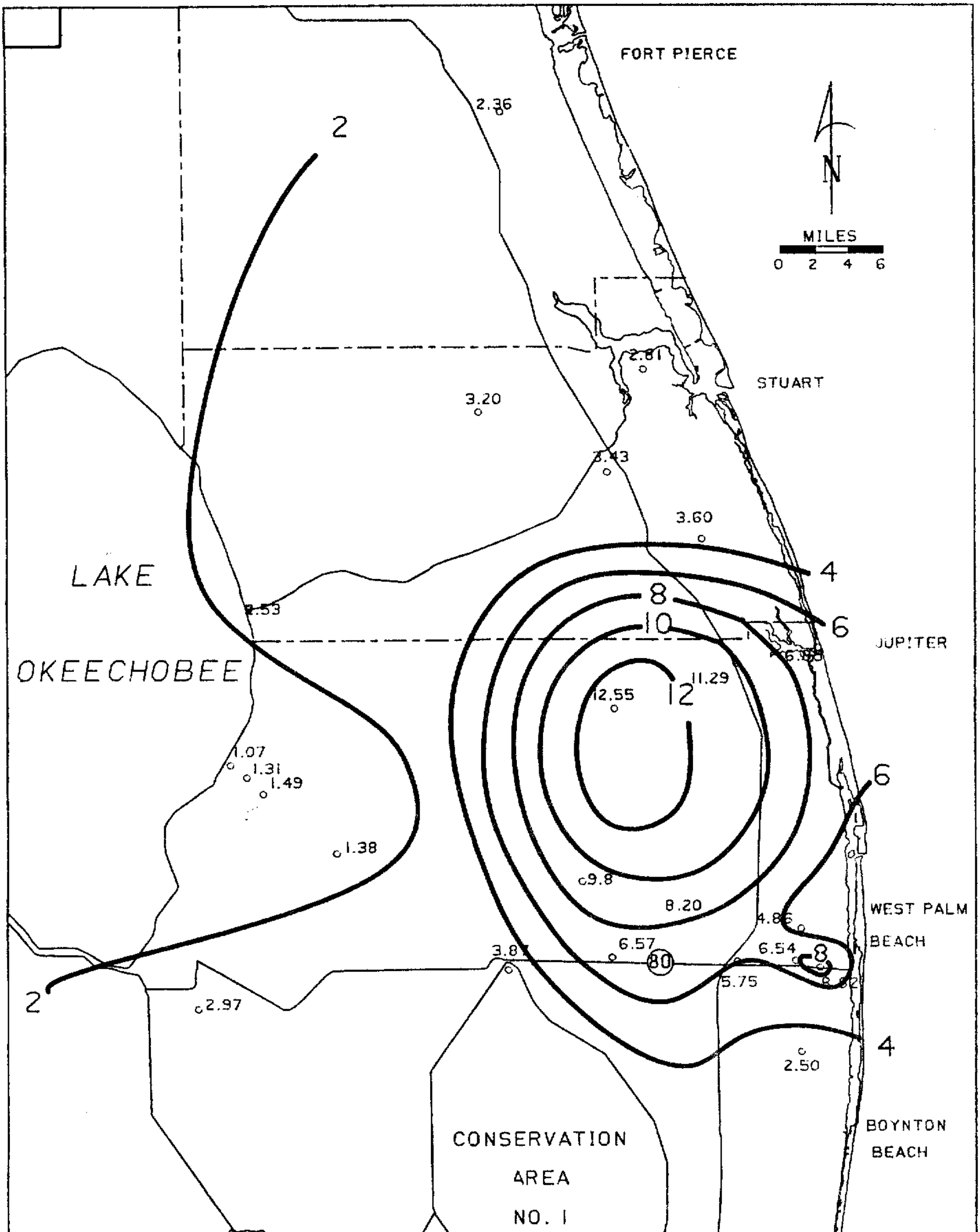

FIGURE 3
ISOHYETAL MAP OF SEPTEMBER 22 - 25, 1983 STORM
IN PALM BEACH AND MARTIN COUNTIES

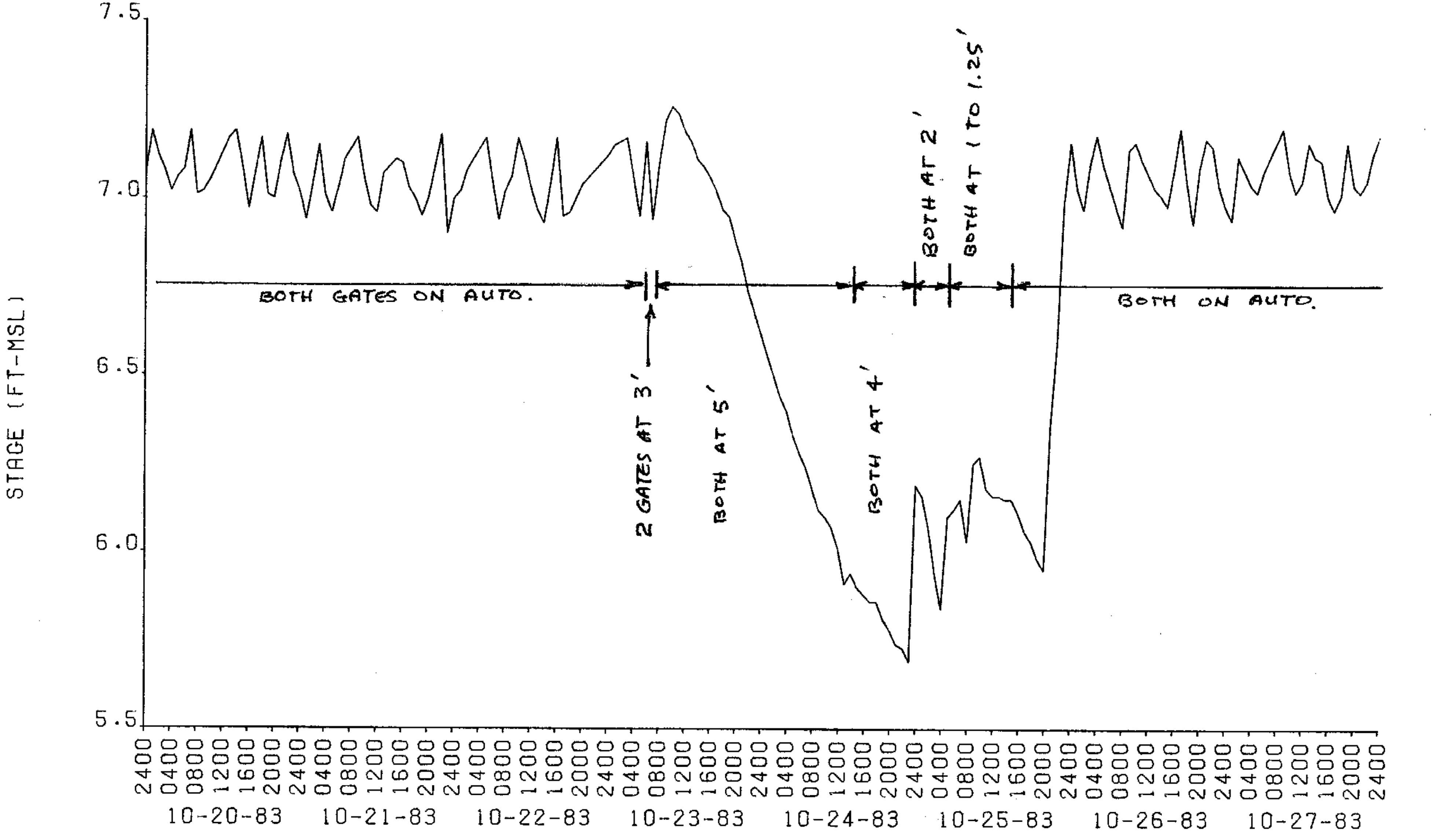

FIG. 4 HOURLY STAGE READINGS AT S-44

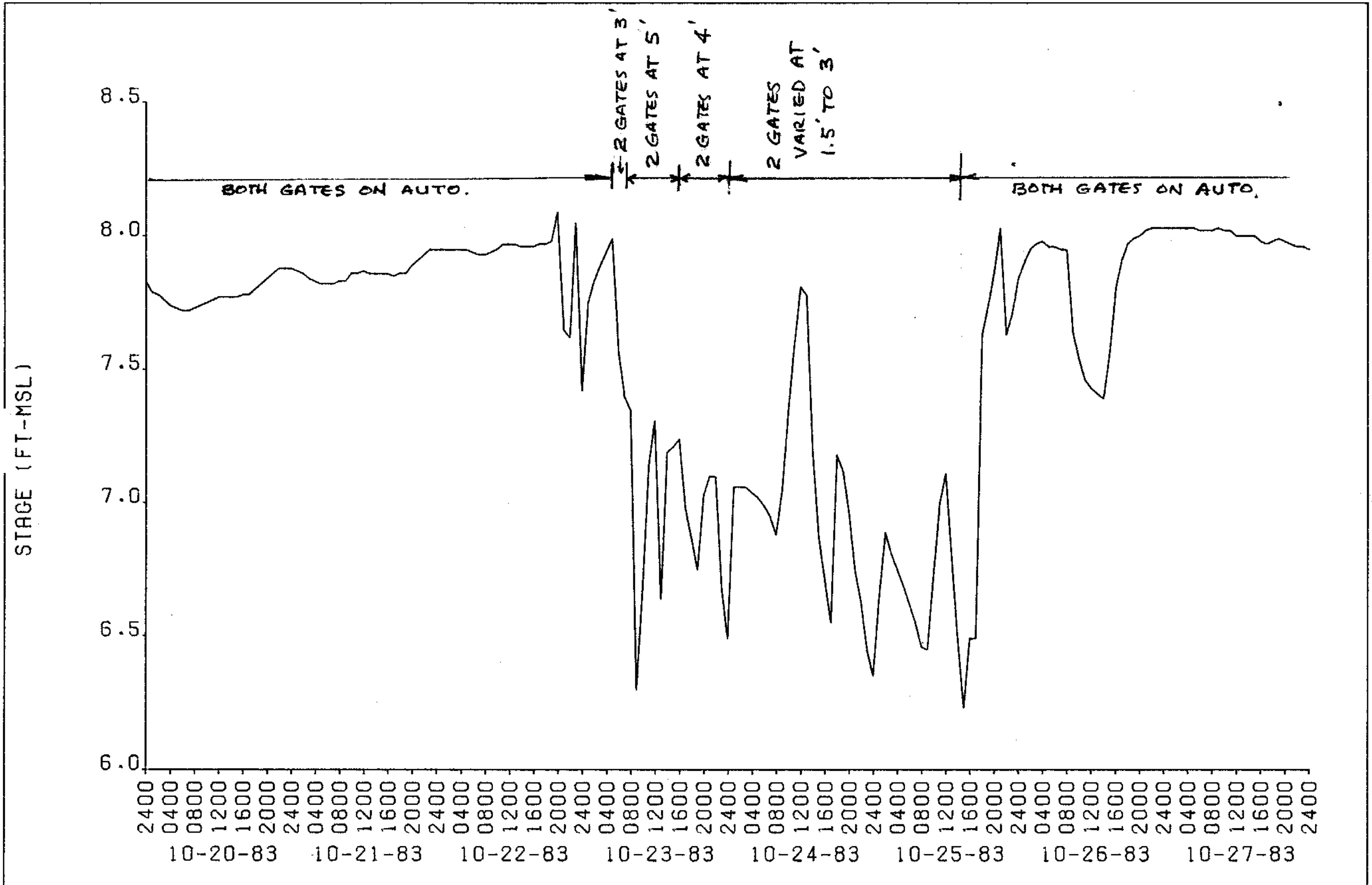

FIG. 5 HOURLY STAGE READINGS AT S-41

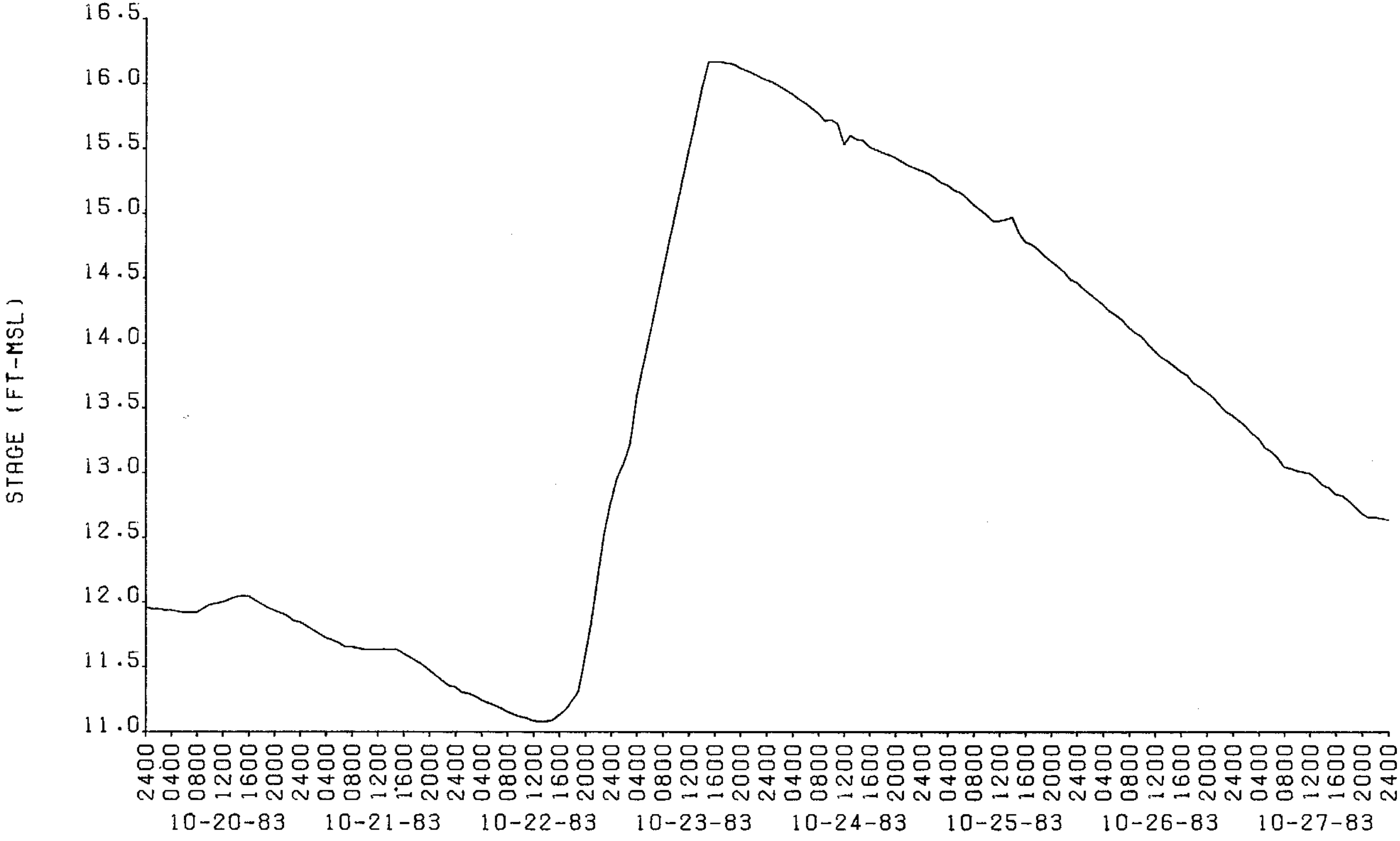

FIG. 6 HOURLY STAGE READINGS AT WELLINGTON BRIDGE

FIG. 7 HOURLY STAGE READINGS AT S-5AE

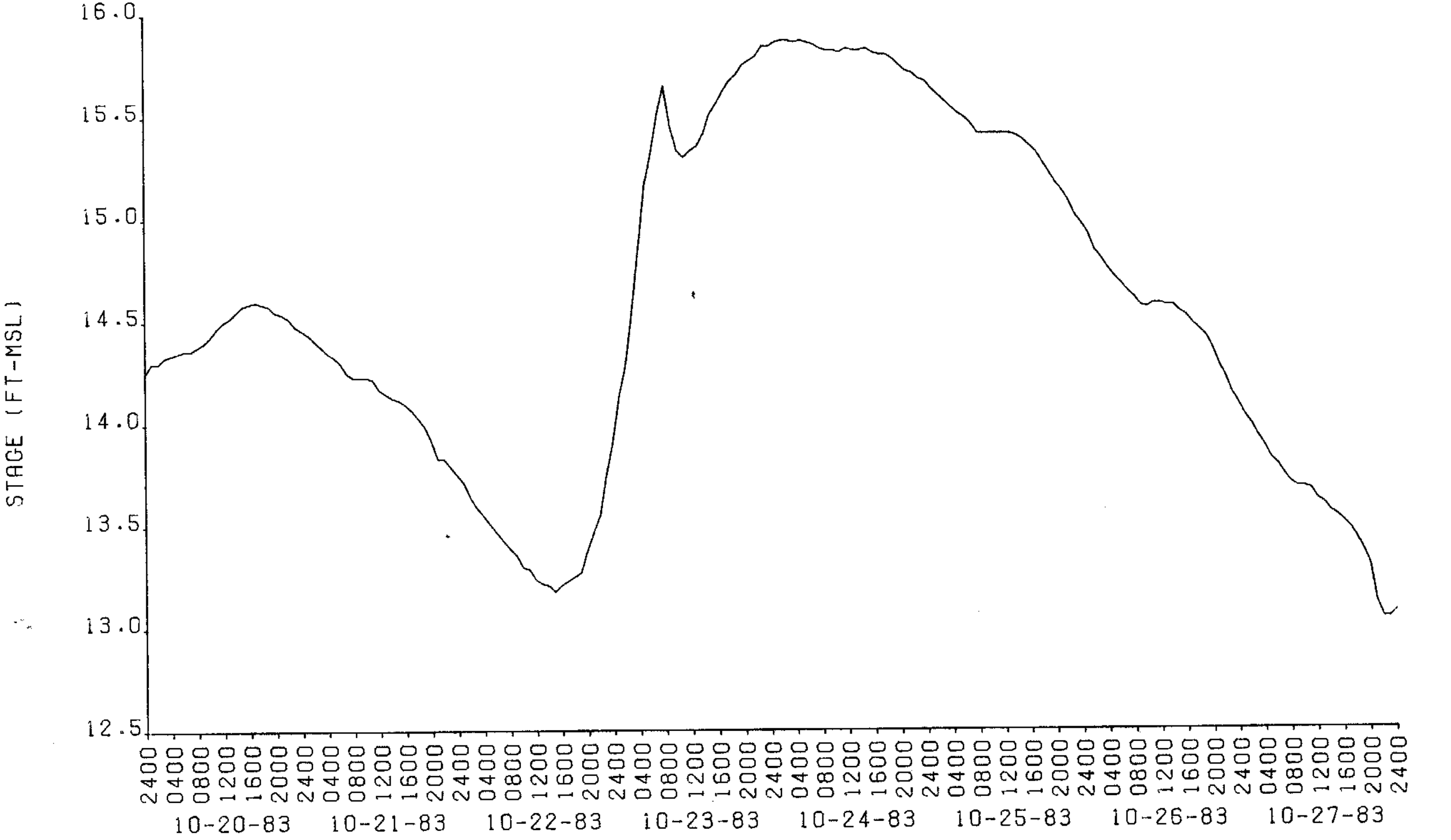

FIG. 8 HOURLY STAGE READINGS AT L-8

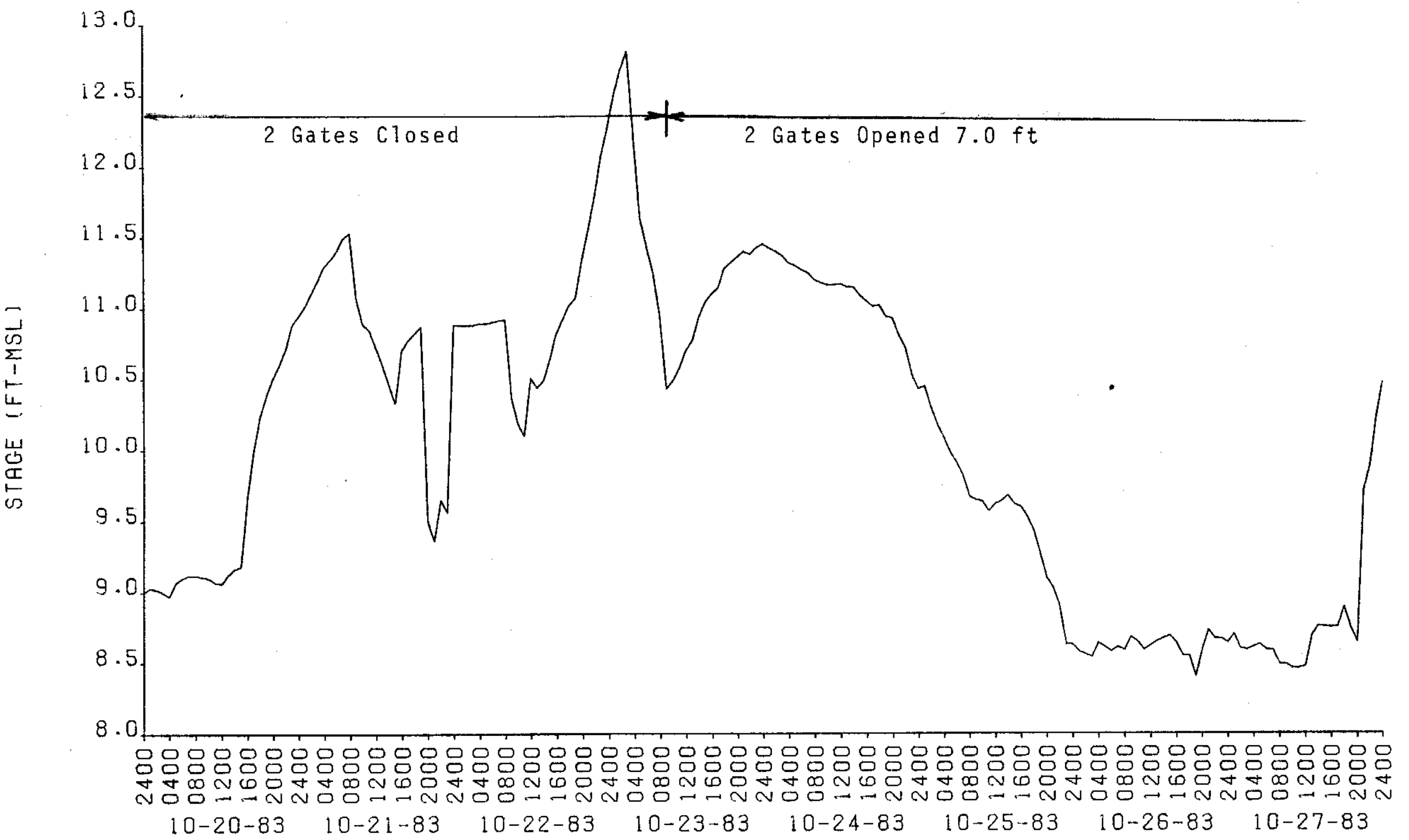

FIG. 9 HOURLY STAGE READINGS AT S-5AW